S&ME

Rhett

January 20, 2003

Mr. Rhett Reidenbach, P.E.
Davis & Floyd, Inc.
3229 West Montague Street
North Charleston, SC 29411

Reference: **ADDENDUM #1 to REPORT OF GEOTECHNICAL EXPLORATION**
Georgetown County Commerce Center
Proposed Pump Station, Sewer Lines, Roadways and Elevated Water Tank
Andrews, South Carolina
S&ME Project No.: 1131-02-651

Dear Mr. Reidenbach:

As requested, I have reevaluated subsurface conditions in the local vicinity of the elevated water tank to determine which IBC 2000 "site class" should be used for its structural design. In our original geotechnical report we stated that the subsurface conditions at the water tank site conform to Site Class E, as determined from one 40-ft deep soil test boring (SB-8). As dictated by the IBC site classification procedures, our site class analysis required an assumption about the properties of the soils between 40 ft and 100 ft. Based on the site location, the IBC 2000 Site Class E designation would dictate that structural design of the tank be based on an acceleration response spectrum developed by performance of a site-specific seismic response analysis.

Recently, we reevaluated the water tank site classification based on some nearby (<1 mile) 95 ft deep borings we performed for an SCDOT bridge project. These borings indicate that deeper (i.e., >40 ft) subsurface conditions in the area are probably better than we originally assumed, and that a Site Class D is more appropriate for the structural design of the tank. Consequently, a site-specific seismic response analysis is not required.

Sincerely,

S&ME, Inc.

Forrest W. Foshee, P.E.
Vice President

FWF/jfc

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REPORT OF GEOTECHNICAL EXPLORATION

GEORGETOWN COUNTY COMMERCE CENTER
PROPOSED PUMP STATION, SEWER LINES, ELEVATED WATER TANK, AND ROADWAYS
ANDREWS, SOUTH CAROLINA
S&ME PROJECT NO. 1131-02-651

Prepared For:

DAVIS & FLOYD, INC.
NORTH CHARLESTON, SOUTH CAROLINA

Prepared By:

S&ME, Inc.
840 Low Country Boulevard
Mt. Pleasant, South Carolina 29464

October 24, 2002

October 24, 2002

Mr. Rhett Reidenbach, P.E.
Davis & Floyd, Inc.
3229 West Montague Street
North Charleston, SC 29411

Reference: **REPORT OF GEOTECHNICAL EXPLORATION**
Georgetown County Commerce Center
Proposed Pump Station, Sewer Lines, Roadways and Elevated Water Tank
Andrews, South Carolina
S&ME Project No.: 1131-02-651

Dear Mr. Reidenbach:

We have completed the geotechnical exploration for the proposed pump station, sewer lines, roadways and elevated water tank at the proposed Georgetown County Commerce Center in Andrews, South Carolina. Our services were provided in general accordance with S&ME Proposal No. 31-02-299, dated September 23, 2002. The purpose of our exploration was to determine the general site subsurface conditions, and then based on our evaluation of those conditions, to provide geotechnical recommendations for the proposed construction. This report presents our understanding of the planned construction, includes a discussion of the site and subsurface conditions, and presents our conclusions and recommendations.

PROJECT INFORMATION

As shown on Figure 1, the proposed Georgetown County Commerce Center is located along U.S. Highway 521 in Andrews, South Carolina. We understand the initial stage of site development will include: 1) approximately 10,900-LF of two-lane and divided four-lane roadways, 2) an 8-ft diameter, 25-ft deep wet well for a submersible pump station, 3) gravity sewer lines, and 4) a 300,000-gallon, elevated water tank with a 70-ft diameter foundation footprint. Also, we understand that the water tank will likely have four to eight support legs.

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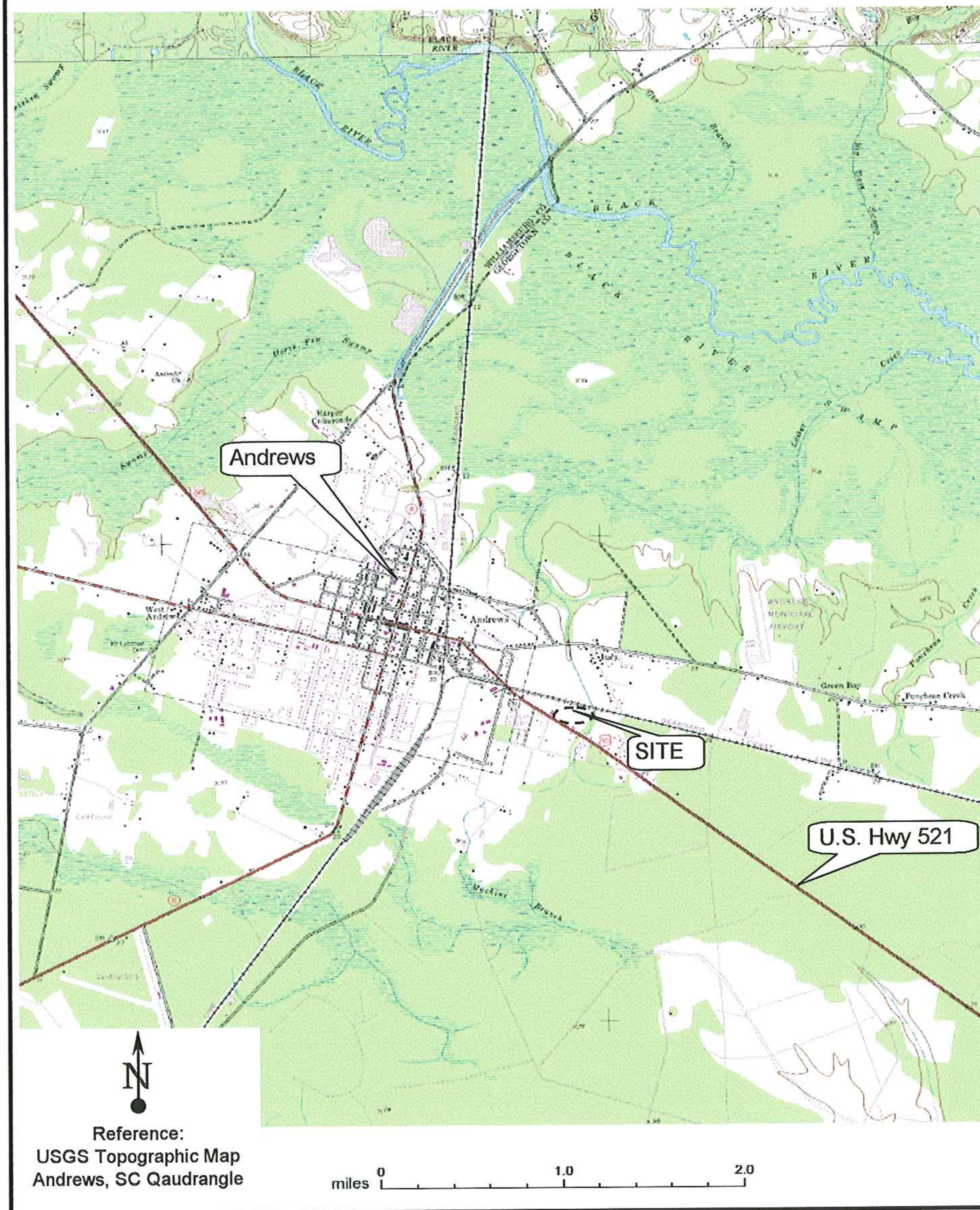


Job No. 1131-02-651	S&ME	SITE LOCATION MAP Georgetown County Commerce Center Andrews, SC	Figure: 1
October 2002			

Figure A in Appendix I shows the locations of the proposed pump station, water tank, roadways and sewer line relative to property boundaries and existing roadways and clearings. We understand that cut and fill amounts of 2 ft or less will be required to grade the proposed roadways, and that little (≤2 ft) or no new fill will be required to grade the proposed water tank area. Also, we understand that the proposed gravity sewer lines will bear 12 ft to 16 ft below grade.

METHODS OF EXPLORATION

Field Drilling and Testing

The site subsurface conditions were explored by performing two cone penetration test (CPT) soundings (designated SB#2 and SB#5) for the proposed sewer line, and two standard penetration test (SPT) borings (designated SB#4 and SB#8) for the proposed pump station and elevated water tank. Soundings SB#2 and SB#5 were terminated at depths of about 25 ft and 30 t below existing grade, respectively, and borings SB#4 and SB#8 were both extended to a depth of 40 ft below existing grade. To explore the shallow subsurface conditions along the proposed roadway alignment, we drilled 18 hand auger borings (designated SB#3, SB#6, SB#7, and SB#9 through SB#22), each to a depth of about 4 ft.

The locations of the CPT soundings, SPT borings and most of the hand auger borings were established in the field using a Global Positioning Satellite (GPS) system with sub-meter accuracy. Due to the heavy tree canopy covering much of the site, some hand auger boring locations in the proposed roadways were established by pacing distances and estimating right angles from existing site features. The approximate test locations are shown on Figure A in Appendix I.

In the CPT soundings (ASTM D 5778), an electronically instrumented cone penetrometer was hydraulically pushed through the soil to measure point stress, porewater pressure and sleeve friction. These values were recorded nearly continuously as the cone was pushed to the desired depth. Using theoretical and empirical relationships, the CPT data was then used to determine soil stratigraphy and to estimate soil parameters such as preconsolidation stress, friction angle and undrained shear strength. Also, a pore pressure dissipation test was performed at a depth of about 10 ft in sounding SB#2. The dissipation test results provide data about the time rate of soil consolidation.

The soil test borings were drilled with a track-mounted drill rig using a mud rotary drilling procedure. Within the borings, Standard Penetration Testing (SPT) and split-spoon sampling were performed at 2½-ft intervals in the upper 10 ft and at 5-ft intervals thereafter. Standard Penetration Tests and split-spoon sampling were performed in general accordance with ASTM D 1586. The split-spoon sampling procedure uses a standard 2-in. O.D. split-spoon sampler that is driven into the bottom of the boring with a 140-pound hammer falling a distance of 30 in. The number of blows required to advance the sampler the last 12 in. of a normal 18-in. penetration is recorded as the Standard Penetration Test (SPT) resistance value (N). These "N"-values, recorded as blows per foot (bpf), are indicated on the boring logs at the depth of occurrence and provide an indication of the relative density of granular materials and the strength of cohesive materials.

At the completion of drilling, the samples were sealed and returned to our laboratory for further examination and classification by an engineer. Samples were classified in the laboratory based on visual observation and limited grain size and plasticity test data. Soil descriptions indicated on the boring logs are in general accordance with the Unified Soil Classification System. Estimated group symbols according to the Unified Soil Classification System are given on the boring logs. A more detailed description of our field testing procedures, and the CPT Sounding

Logs, SPT Boring Logs, Hand Auger Boring Logs and Dissipation Test Results are included in Appendix I.

SITE AND SUBSURFACE CONDITIONS

Site Conditions

The proposed Georgetown County Commerce Center is located between U.S. Highway 521 and the Seaboard Coast Line Railway in Andrews, South Carolina. As shown on Figure A in Appendix I, the proposed pump station is located near the center of the proposed development, and the proposed water tank is located at the southern corner of the proposed development. The majority of the site is wooded with young to mature pines and hardwoods; however, at the time of our exploration, the northwest portion of the site had been cleared. Several gravel roads and cleared paths traverse the site. Relatively shallow (about ≤2-ft deep) ditches were located adjacent to most of the gravel roads. Additionally, an approximately 5-ft deep ditch was located in the right-of-way along U.S. Highway 521. The ditches along the gravel roadways and along U.S. Highway 521 contained some water at the time of our exploration.

At the time of our exploration, much of the site was covered with standing water, which indicates the presence of shallow, low-permeability, clayey soils which impede rainfall infiltration. Photographs of several areas with standing water are included as Figure B in Appendix I. Based on the topographic information on the USGS Andrews Quadrangle map, the site elevation ranges from approximately 26 ft-MSL[1] to 30 ft-MSL.

[1] Mean Seal Level (feet)

Soil Survey Data

The USDA Soil Conservation Service's (SCS) *Soil Survey of Georgetown County, South Carolina*, dated December, 1982 (excerpt shown on the Soil Survey Map, Figure C, in Appendix I) indicates that the majority of the site contains the following SCS Soil Series: Eulonia (26A), Wahee (59) and Bladen (13). Some of the more pertinent USCS information on these soil series is summarized in the table below.

Table 1. Summary of Selected USCS Soil Series Information

SOIL SERIES	CLASSIFICATION (USCS/AASHTO)	SEASONAL HIGH WATER TABLE	DRAINAGE	FILL SUITABILITY	ROADWAY DEVELOPMENT	SHALLOW EXCAVATION LIMITATIONS
Eulonia (26A)	SM, SM-SC/A-2 SC, CL/A-6, A-7, A-4	1.5-3.5 ft	Favorable	Fair, wetness	Moderate: low strength, wetness	Severe: wetness
Wahee (59)	SM, SM-SC/A-2, A-4 CL, CH/A-7, A-6	0.5-1.5 ft	Percs slowly	Poor, low strength, wetness	Severe: low strength, wetness	Severe: wetness
Bladen (13)	CL, ML/A-4 CH/A-7	0-1 ft	Percs slowly	Poor, low strength, wetness	Severe: low strength, wetness	Severe: wetness

As shown in Table 1, the USCS evaluations indicate that the Eulonia (26A) soil series is fair for fill suitability, favorable for drainage and presents "moderate" limitations to roadway development. The Wahee (59) and Bladen (13) soil series are described as poor for fill use, they perc slowly, and present "severe" limitations for roadway development. In general, the difference between the Eulonia series and the Wahee and Bladen series is the fines content. Soils with a large fraction of silt or clay particles will behave as cohesive soils. A relatively high water table is expected in all of the soil series, and cohesive soils are more sensitive to moisture and changes in moisture. Although cohesive soils can often serve the same function as cohesionless soils (e.g., as fill or pavement subgrades), the cohesive soils are typically much more difficult to work with. Additionally, very wet cohesive soils are often relatively weak and compressible.

Subsurface Conditions

Details of the subsurface conditions encountered by the CPT soundings, SPT borings and hand auger borings are shown on the logs in the Appendix. The logs represent our interpretation of the subsurface conditions based upon the CPT data and visual examination of the split spoon samples and auger cuttings. Stratification lines on the logs represent approximate boundaries between soil types; however, the actual transition may be gradual. The general subsurface conditions and their pertinent characteristics are discussed in the following paragraphs.

Pump Station. Boring SB#4, which was performed for the proposed pump station, initially encountered about 8-in. of topsoil. The topsoil was underlain by firm to stiff clay to a depth of about 14 ft, and then medium dense sand to the top of the Santee Limestone at a depth of about 25 ft. The clay had SPT N-values ranging from 5 blows per foot (bpf) to 9 bpf, and the sand had N-values of 11 bpf and 14 bpf. The limestone was sampled as medium dense, calcareous sand and continued to the 40-ft boring termination depth. N-values in the limestone ranged from 21 bpf to over 100 bpf. We note that the top of the limestone formation is at the planned depth of the pump station; therefore, some difficult excavation might be encountered.

Sewer Line. Soundings SB#2 and SB#5, which were performed for the proposed gravity sewer line, encountered interbedded layers of very loose to medium dense sand and soft to firm clay and silt to the top of the Santee Limestone. The limestone, which was encountered at depths of about 23 ft in SB#2 and 27 ft in SB#5, was very dense, with cone tip resistances in excess of 300 tsf. At the 12-ft to 16-ft sewer line bearing depth, the soils will likely consist of loose, clayey sand or soft to firm clay and silt.

Elevated Water Tank. Boring SB#8, which was performed for the proposed elevated water tank, initially encountered about 3 in. of topsoil. Beneath the topsoil, the boring encountered very soft to firm clay to the top of the Santee Limestone, which was encountered at a depth of about 25 ft below existing grade. The clay strata was interbedded with an approximately 2½-ft thick layer of medium dense sand at a depth of about 7½ ft. N-values in the clay ranged from less than 1 bpf to 5 bpf, and the medium dense sand had a N-value of 20 bpf. The limestone continued to a depth of about 38½ ft and was sampled as loose to dense, calcareous sand with N-values ranging from 5 bpf to 34 bpf. From a depth of about 38½ ft to the boring termination depth (40 ft), medium dense, silty sand with a N-value of 29 bpf was encountered.

Roadway Alignment. The hand auger borings along the proposed roadway alignment initially encountered about 2½ in. to 12 in. of topsoil. Beneath the topsoil, the hand auger borings generally encountered clay and silt to the termination depth (4 ft). However, hand auger borings SB#1 and SB#9, and sounding SB#2, which are located at the western end of the proposed roadway, encountered clayey sand beneath the topsoil to the 4-ft termination depth of the hand auger borings, and to a depth of about 10 ft in SB#2.

Water Depth. At the time of our exploration, groundwater was measured within the sounding and soil test boring holes at depths ranging from about 2 ft to 5 ft below existing grade. Within the hand auger borings, groundwater was encountered at the ground surface to a depth of about 2½ ft. We note that water levels will fluctuate due to seasonal and climatic variations, and with construction activity in the area. Additionally, as evidenced by the ponded water across the site, the surficial clayey soils will impede rainfall infiltration. Therefore, the possibility of groundwater fluctuations should be considered when finalizing the design and construction plans for this project.

CONCLUSIONS AND RECOMMENDATIONS

The analyses and recommendations submitted herein are based, in part, upon data obtained from our subsurface exploration. The nature and extent of subsurface variations between the CPT soundings, SPT borings, and hand auger borings will not become evident until construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, location or depth of the proposed structures or roadways are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing. We strongly recommend that S&ME be retained to review the final design plans and specifications to confirm that earthwork and foundation recommendations are properly interpreted and implemented.

Pump Station and Sewer Lines

Pump Station. Boring SB#4, which was drilled at the proposed pump station, indicates that the bearing surface at a depth of about 25 ft below existing grade will be limestone. Therefore, the subsurface conditions will be suitable for support of the pump station on a shallow mat foundation. We estimate that total settlement of the pump station bearing on the limestone will be negligible (i.e., ≤½ in.).

Since limestone is present at the wet well bearing level, we do not anticipate that overexcavation and backfilling with stone will be necessary to stabilize the excavation. However, excavation of the limestone (if necessary to reach planned subgrade elevation) may be difficult. We anticipate that the excavation for the proposed wet well will be performed within the confines of a temporary sheet-pile wall or similar retaining system. Due to the size of the excavation, the shallow groundwater level, and the presence of limestone at the bearing level, the retaining system and excavation methodology for the wet well should be designed and developed by a knowledgeable engineer/contractor team.

Sewer Lines. Soundings SB#2 and SB#5, which were performed for the proposed gravity sewer line, encountered either soft cohesive soils or medium dense clayey sand at the planned 12-ft to 16-ft bearing depth. Since the installation of the proposed sewer lines will not result in a significant stress increase (i.e., the weight of the soil removed is likely greater than the weight of the utility that replaces it), we anticipate that total settlements of the sewer line will be less than 1 in. Differential settlements are estimated to be about half of the total settlement. However, to provide a uniform bearing surface and maintain stability of the excavation bottom during construction, we recommend that at the foundation bearing level, the soils be overexcavated at least 12 in. and replaced with coarse aggregate. Depending on the conditions encountered at the time of excavation, additional excavation may be necessary. In very soft bottom areas, reinforcing the soil subgrade with a geotextile will reduce the required thickness of the stone layer.

Dewatering. The excavations for the pump station and sewer line installation will encounter groundwater. Water levels should be maintained at least 2 ft below the excavation bottoms throughout construction to maintain bottom stability. This can probably best be accomplished with a temporary well point system, but possibly by pumping from sumps located within the excavations. The dewatering system should be designed by a qualified dewatering contractor.

Excavations. All excavations should be sloped or shored in strict compliance with the most recent local, state, and federal governing regulations, including OSHA (29 CFR Part 1926) excavation trench safety standards. Stockpiles should be placed well away from the edge of the excavation and their height should be controlled so they do not surcharge the sides of the excavation. The responsibility for excavation safety and stability of temporary construction slopes and shoring should lie solely with the contractor. This information is provided only as a service and under no circumstance should we be assumed responsible for construction site safety.

Lateral Earth Pressure. The pressure on below-grade walls is a function of the relative movement between the structure and the surrounding soils. Three idealized conditions of lateral earth pressure (active, passive and at-rest) have been used in classical soil mechanics and are commonly used for design. Active earth pressure occurs when the wall moves away from the soil and the soil mass stretches horizontally, fully mobilizing its shear strength, until a condition of plastic equilibrium is reached. Passive earth pressure occurs when a soil mass is compressed horizontally, fully mobilizing its shear resistance. A soil mass that is neither stretched nor compressed is said to be in an "at-rest" state.

Below-grade walls for the pump station should be designed as rigid and restrained against rotation; thus, the walls should be designed for an "at-rest" condition. Since the backfill soils will likely be the on-site clays and clayey sands, we recommend that all below grade walls be designed for an at-rest earth condition. We recommend that an equivalent fluid weight of about 100 lb/ft^3 be used. The design of the below grade walls (for the pump station) should take into account the hydrostatic pressures resulting from differential water levels between the interior and exterior of the wet well. The wall backfill should be compacted to 90% of the soil's Modified Proctor maximum dry density (ASTM D 1557) to help limit lateral stress. We also recommend that self-propelled compaction equipment not be used within 5 ft of the walls.

We recommend that all temporary shoring or bracing for the proposed sewer lines be designed to resist lateral earth pressures. The following soil parameters may be used to estimate lateral earth pressures for the on-site clayey soils for the design of temporary shoring or bracing.

On-Site Clays	
Internal Friction Angle (ϕ)	22°
Cohesion (psf)	400 psf
Moist Unit Weight (pcf)	120 pcf
Submerged Unit Weight (pcf)	58 pcf
At-rest Earth Pressure (K_o)	0.62
Active Earth Pressure (K_a)	0.45
Passive Earth Pressure (K_p)	2.22

Note: K_o, K_a, and K_p are for level backslope and no wall friction.

Elevated Water Tank

Because of the soft compressible clays encountered in the upper 25 ft of boring SB#8, we recommend that the proposed water tank be supported on deep foundations bearing in the limestone. The limestone was encountered in the boring at a depth of about 25 ft below existing grade.

Typical deep foundation systems for water tank structures consist of a group of driven piles below each leg support. The three most common types of driven piles are prestressed concrete (PSC), timber and steel H-piles. For the subsurface conditions at this site, there are advantages and disadvantages associated with each of these pile types. We anticipate that timber or prestressed concrete piles will likely achieve sufficient axial compressive capacity if they are driven to virtual refusal (i.e., 20 blows per inch) in the limestone. However, because there is variability in the limestone (i.e., it is not a homogeneous or predictable stratum), the refusal depth will likely vary from pile to pile. This would result in various pile "stick up" lengths above the ground surface, and the contractor would have to cut piles to accommodate the actual pile driving refusal depth.

Cutting off timber piles would be easier and cheaper than cutting off PSC piles; however, PSC piles have significantly more capacity than timber piles (i.e., greater number of timber piles required for same capacity as smaller number of PSC piles). The steel H-piles would also be easier than PSC piles to cut off; however, we are doubtful that the steel H-piles would achieve sufficient axial capacity in the limestone (i.e., steel H-piles might not refuse in the limestone, but easily penetrate to greater depths). Because of this uncertainty, we have not provided recommendations for steel H-piles. Recommendations for driven 12-in. and 14-in. square PSC piles and 10-in. tip diameter timber piles are provided hereinafter. With either pile type, testing should be performed during driving to confirm the axial compressive capacity.

To avoid the uncertainty associated with various pile lengths, cutoff amounts, etc., the proposed water tank could be supported on drilled, cast-in-place foundations. Drilled, cast-in-place foundations such as drilled shafts or steel micro-piles could be installed to a planned depth, allowing for more accurate foundation cost estimates prior to construction. Also, because drilled shafts are larger in diameter than driven piles, they have greater axial and lateral capacities, allowing for one foundation beneath each leg support and elimination of a pile cap. We have performed axial and lateral analyses of 30-in., 36-in., 42-in. and 48-in. diameter drilled shaft foundations.

Driven Pile Foundations

Axial Compressive Capacity. We have estimated allowable axial compressive capacities for 10-in. tip diameter timber piles and 12-in. and 14-in. square PSC piles. We anticipate that timber and PSC piles driven to virtual refusal in the limestone will achieve the allowable axial compressive capacities shown in Table 2. Based on the boring, we anticipate that refusal will be encountered between depths of about 27 ft and 35 ft below existing grade.

Table 2. Allowable Axial Compressive Capacities for Piles Driven To Refusal In Limestone

Pile Type	Allowable Axial Compressive Capacity
10-in. tip diameter timber pile	35 tons
12-in. square PSC pile	60 tons
14-in. square PSC pile	75 tons

An efficiency factor (to account for capacity reductions caused by group effects) of 1.0 should be used for center-to-center pile spacings of five pile diameters or more. Factors decrease linearly to 0.80 for a spacing of three pile diameters, which is the minimum recommended spacing. The structural capacity of the piles has not been considered in our analysis. Allowable tensile capacities are estimated to be approximately one-third of the allowable compressive capacities.

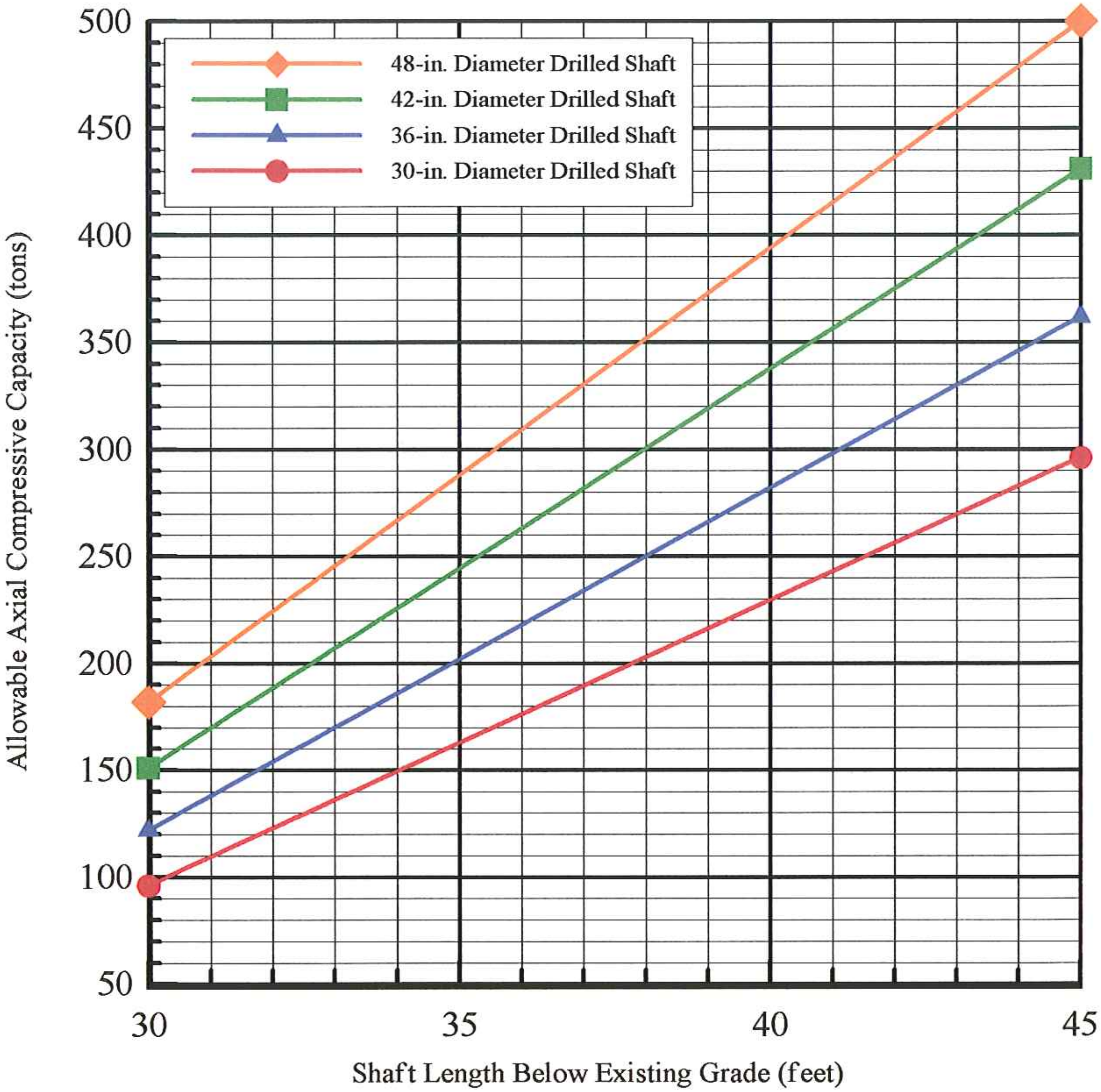
Pile Installation. Based on our experience with similar projects, drop, air, or diesel hammers having rated energies in the range of 30 to 60 ft-kips should be suitable for pile installation. However, final hammer approval should be based on a wave equation analysis that accurately reflects the contractor's proposed driving system. The results of the wave equation analysis will confirm that the proposed driving system is capable of driving the piles to the necessary capacity without causing any damage to the pile. We recommend that driving shoes be used to limit any damage to the pile toe. Pre-auguring may be performed in the upper 25 ft, provided the diameter of the auger is smaller than the least pile dimension. Jetting should be prohibited.

Pile Testing. The uncertainty in depth versus capacity (and the practical refusal depth) for driven piles can be eliminated by dynamically testing several probe piles immediately prior to construction. We recommend that three probe piles be driven in the area of the proposed tank (at production pile locations) before production piles are ordered, to confirm the required pile length (realizing that pile "stick up" may result from various refusal depths). The installation of these piles should be monitored with a Pile Driving Analyzer™ (ASTM D 4945) during installation. Using the results of the dynamic testing, the pile capacity can be verified. Not only will the Pile Driving Analyzer be effective in verifying that the capacity is achieved, it will also measure pile integrity, hammer performance and stresses within the pile. By measuring the stresses within the pile, we can determine if the pile is in danger of being damaged (integrity of the pile), which may become a problem in hard driving. Using the hammer performance measurements, we can perform a refined wave equation analysis and produce an accurate driving criterion (i.e., how many blows to achieve a specified capacity). An engineering technician, under the direction of the geotechnical engineer, should monitor all pile driving to verify that the piles are encountering the expected driving resistances and note any damage to the pile.

Drilled Shaft Foundations

Axial Compressive Capacity. Figure 2 shows the estimated allowable axial compressive capacities for drilled shaft foundations. The structural capacity of the shafts has not been considered in our analysis. Allowable tensile capacities are estimated to be approximately two-thirds of the allowable compressive capacities.

Shaft Installation. Due to the relatively shallow groundwater depth, we recommend that the "slurry" method be used for shaft construction. Additionally, the shaft excavation may be stabilized by installing a temporary steel casing. The temporary casing is typically installed and removed with a vibratory hammer. Slurry drilled shafts are constructed by conventional caisson drill rigs excavating beneath a drilling mud slurry. Typically, the slurry is introduced into the excavation after the groundwater table has been penetrated and/or the soils on the sides of the excavation are observed to be caving-in. When the design shaft depth is reached, fluid concrete is placed through a tremie pipe at the bottom of the excavation. The slurry level should be maintained at a minimum of 5 ft or one shaft diameter, whichever is greater, above the groundwater table.



NOTES:

1) The structural capacity of the shafts has not been considered in our analysis and should be evaluated by the project structural engineer.

JOB # 1131-02-651 OCTOBER 2002 NOT TO SCALE	S&ME	ALLOWABLE COMPRESSIVE CAPACITY WATER TANK GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA	FIGURE 2

The bottom of the drilled shaft excavation should be free of debris and loose soil. Concrete should be placed with a tremie tube, and the tube should not be withdrawn from the concrete during the pour. During construction, the casing removal rate should be regulated to maintain a positive pressure head of concrete. New shafts should not be drilled within six shaft diameters of "green" shafts (i.e., shafts less than 24 hrs old).

Shaft Inspection. Inspection during excavation should include verification of plumbness, maintenance of sufficient slurry head, monitoring the specific gravity, pH and sand content of the drilling slurry, and monitoring the depth of the excavation. The specific gravity or relative density of the drilling mud slurry should be monitored from the initial mixing to the completion of the excavation. An increase in the specific gravity or density of the drilling slurry by as much as 10 percent is indicative of soil particles settling out of the slurry onto the bottom of the excavation. This settling will result in a reduction of the allowable bearing capacity of the bottom of the drilled shaft, unless cleaned immediately before placing concrete.

Shaft Testing. The allowable axial compressive capacities shown in Figure 2 are based on a factor of safety of 3.0. Since this factor of safety is relatively conservative, and a load test program for drilled shafts is complex and expensive, a drilled shaft load testing program is not required for this project. However, if a load test program were performed, a factor of safety of 2.0 could be justified. If desired, we will provide recommendations for a drilled shaft load test program for this project.

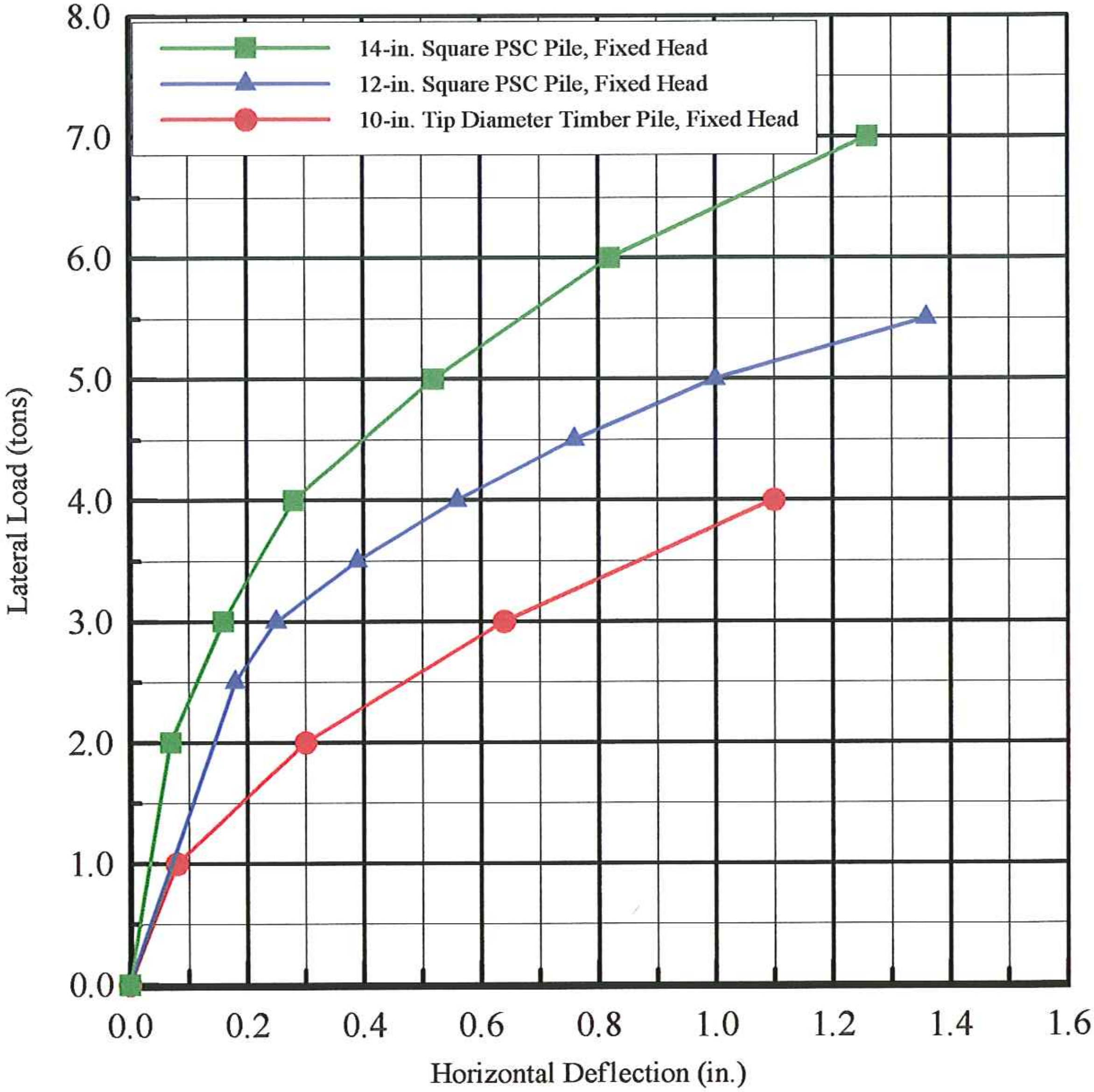
Lateral Foundation Analyses

We performed lateral analyses for single piles and drilled shafts using the proprietary computer program LPILE$^{PLUS©}$.[2] This program performs a beam-column analysis of single piles/shafts, which are subjected to given lateral and axial loading, and assumes a non-linear soil response. Our estimates of lateral load versus horizontal pile head deflection, and lateral load versus maximum bending moment, are presented in Figures 3 through 6. For the driven piles, we assumed a fixed pile head condition for our analysis, and a free head condition was assumed for our analysis of drilled shafts. The deflection and moment versus depth graphs for various lateral loads are included in Appendix II.

We anticipate that if driven piles are used, they will be configured in pile groups beneath the individual leg supports. In our lateral analyses of individual piles, we included a slight reduction factor to account for the effects of "soil shadowing" when piles are in a group. However, the actual pile group configuration will have a significant effect on the behavior of the piles under lateral loading. Therefore, once the pile type and size, loading conditions, and direction of loading are determined, we should be retained to reanalyze the lateral load behavior of the pile groups, and make appropriate revisions.

[2] Reese, Lymon C., Wand, Shin-Tower, LPILE^{PLUS}, Version 1.0, Ensoft, Inc., 1993.

LATERAL LOAD VERSUS HORIZONTAL DEFLECTION FOR DRIVEN PILES
FIXED HEAD CONDITION

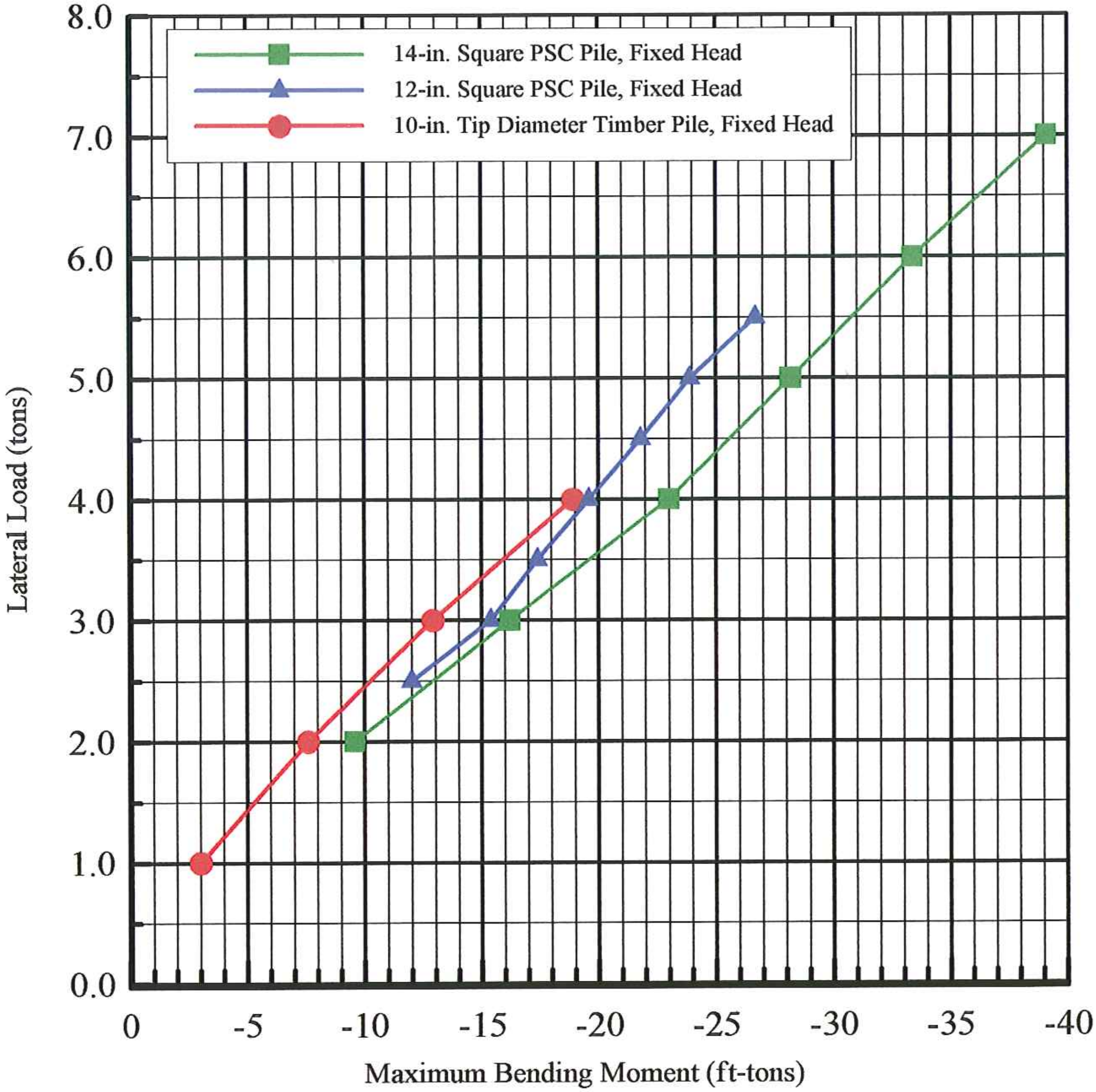


NOTES:

1) The driven PSC piles were assumed to have an effective prestress of 700 psi.
2) The 12-in. square PSC pile was assumed to have four ½-in. diameter steel reinforcement strands.
3) The 14-in. square PSC pile was assumed to have eight 7/16-in. diameter steel reinforcement strands.
3) The lateral analyses were performed for single piles with a slight reduction factor to account for "soil shadowing" that occurs when piles are in a group. Once the foundation type, loading conditions and direction of loading are determined, we should be retained to reanalyze the behavior of the pile groups under lateral loading.

JOB # 1131-02-651 OCTOBER 2002	S&ME	LATERAL CAPACITY WATER TANK GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA	FIGURE 3

LATERAL LOAD VS. MAXIMUM BENDING MOMENT FOR DRIVEN PILES FIXED HEAD CONDITION



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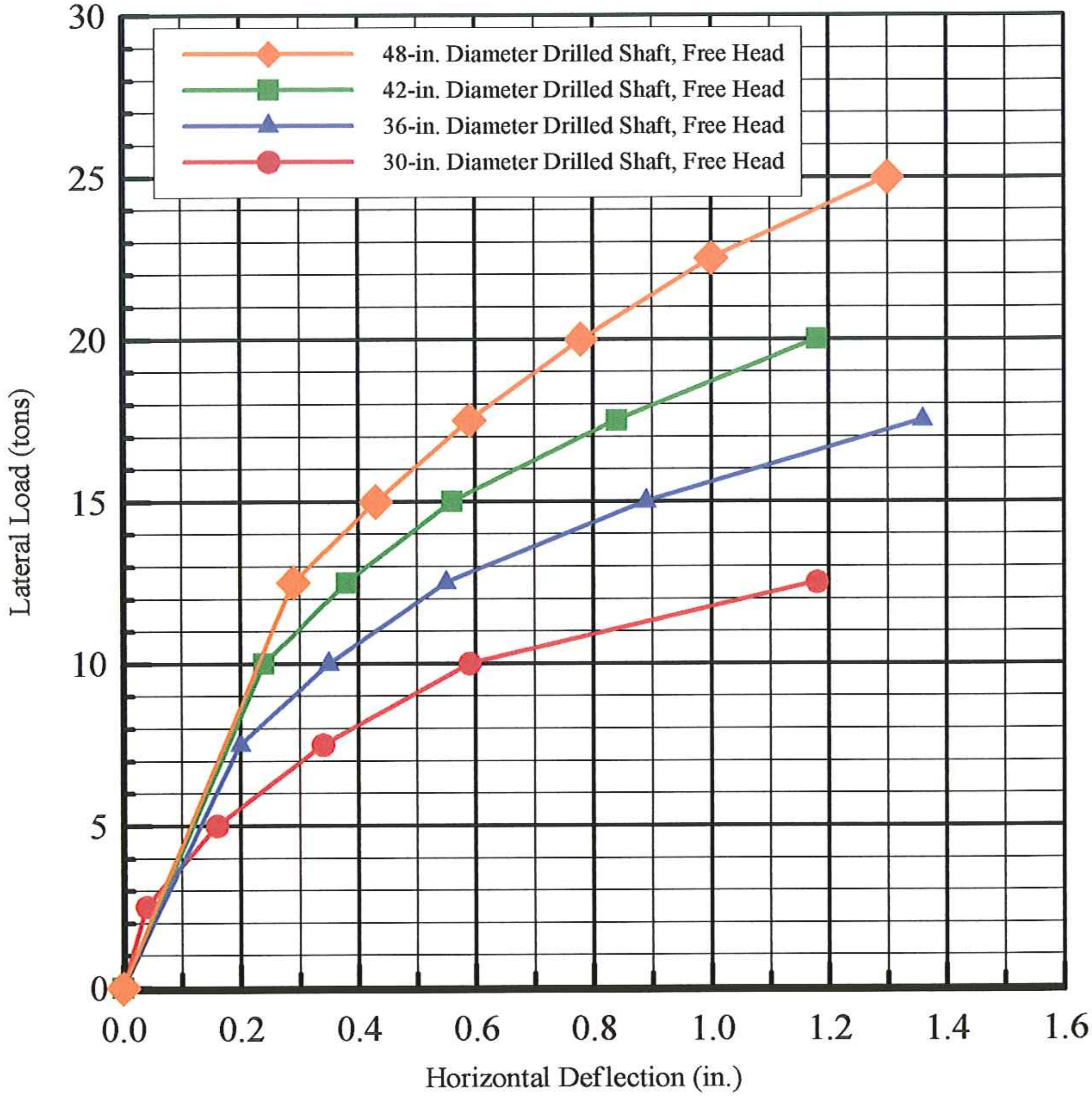
3) The 14-in. square PSC pile was assumed to have eight 7/16-in. diameter steel reinforcement strands.
3) The lateral analyses were performed for single piles with a slight reduction factor to account for "soil shadowing" that occurs when piles are in a group. Once the foundation type, loading conditions and direction of loading are determined, we should be retained to reanalyze the behavior of the pile groups under lateral loading.

JOB # 1131-02-651 OCTOBER 2002	S&ME	LATERAL CAPACITY WATER TANK GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA	FIGURE 4

LATERAL LOAD VS. DEFLECTION FOR DRILLED SHAFTS
FREE HEAD CONDITION

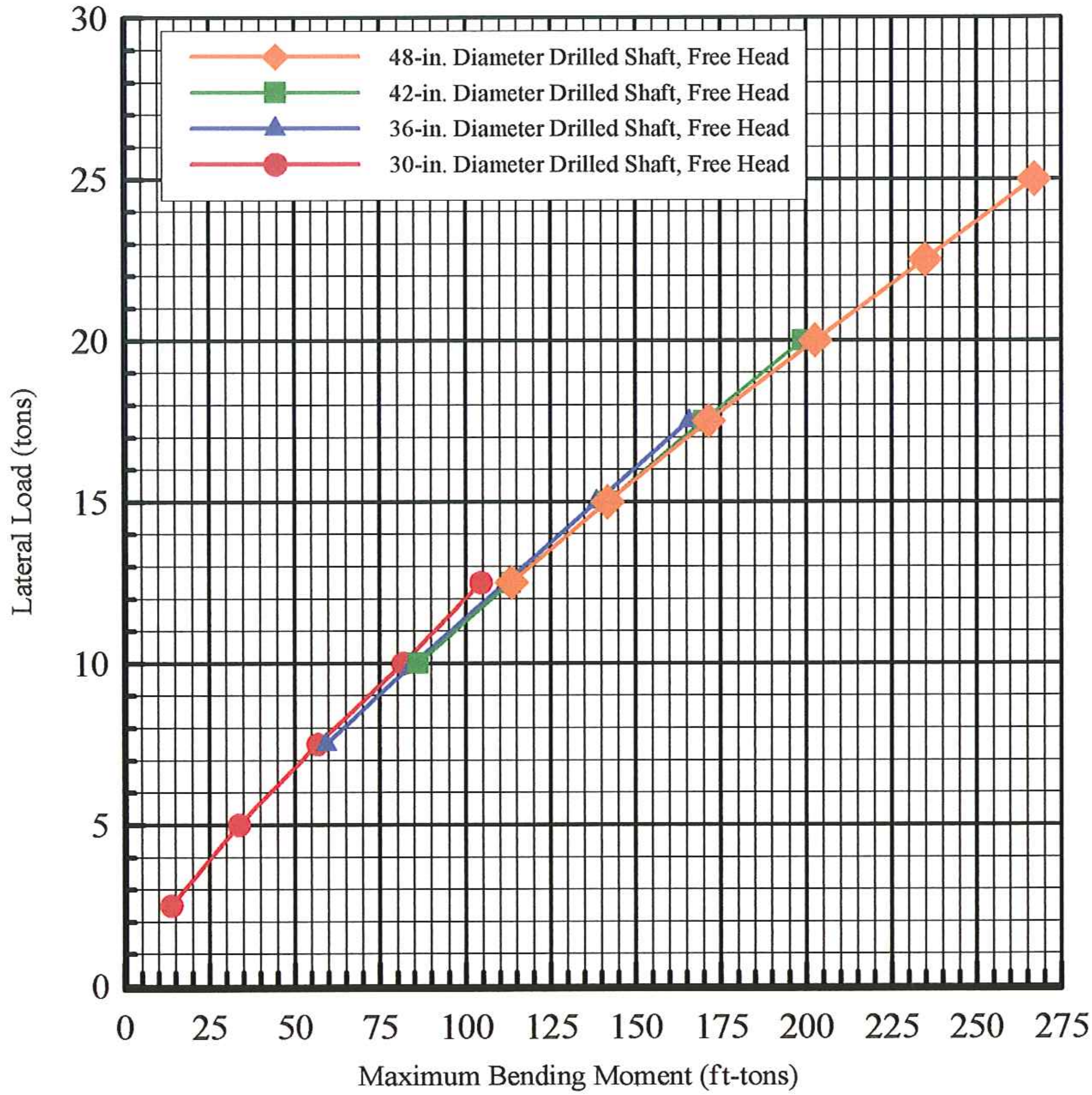


NOTES:

1) The area of steel reinforcement in the drilled shafts was assumed to be approximately 1% of the shaft's cross-sectional area.
2) Shaft heads were assumed to be free.

JOB # 1131-02-651 **OCTOBER 2002**	S&ME	**LATERAL CAPACITY** **WATER TANK** **GEORGETOWN COUNTY COMMERCE CENTER** **ANDREWS, SOUTH CAROLINA**	**FIGURE** **5**

LATERAL LOAD VS. MAXIMUM BENDING MOMENT FOR DRILLED SHAFTS
FREE HEAD CONDITION



NOTES:

1) The area of steel reinforcement in the drilled shafts was assumed to be approximately 1% of the shaft's cross-sectional area.
2) Shaft heads were assumed to be free.

JOB # 1131-02-651 **OCTOBER 2002**	**S&ME**	**LATERAL CAPACITY** **WATER TANK** **GEORGETOWN COUNTY COMMERCE CENTER** **ANDREWS, SOUTH CAROLINA**	**FIGURE** **6**

Seismic Considerations

The subsurface conditions encountered by boring SB#8 (drilled for the proposed elevated water tank) were evaluated in accordance with the procedures of the 2000 International Building Code (IBC). A medium dense, silty sand layer from a depth of about 7½ ft to 10 ft is potentially liquefiable[3] during the design earthquake[4]. Liquefaction of this layer could cause about 1 in. of volumetric compression (or seismically-induced settlement) during or after the design earthquake. Volumetric compression occurs as seismically-induced pore water pressures which build up during an earthquake dissipate. However, since the proposed water tank will be supported on deep foundations bearing below the potentially liquefiable layer, the risk of volumetric compression will be mitigated. Since the potentially liquefiable layer is relatively thin and not expected to significantly influence the overall response of the site during the design earthquake, and since the boring encountered a soft, cohesive layer greater than 10 ft in thickness, the subsurface conditions are classified as a Site Class E. The structural design may be based on a Site Class E.

Roadways

Site Drainage. As previously stated, much of the proposed roadway alignment was covered by standing water at the time of our exploration (i.e., site drainage is poor). Therefore, prior to beginning mass clearing and grading, it is critical that positive site drainage be established. Drainage improvements should be made to lower the water level at the site and handle any rainfall during construction. Site drainage can be established by improving the existing swales and excavating additional gravity draining ditches across the site to divert water flow away from

[3] Liquefaction, the loss of a soil's shear strength due to the increase in porewater pressure resulting from seismic vibrations, is always a potential concern in coastal South Carolina.

[4] The IBC design earthquake has a 98% probability of non-exceedance in 50 years, which is based on an earthquake with a magnitude of 7.3 and "base" acceleration of 0.60g.

construction areas. Depending on conditions at the time of construction, pumping from sumps may be required in low-lying areas of the site. Even during dry weather conditions, ditches and drainage improvements should be in place to handle any heavy rainfall that might occur during construction.

Site Preparation. After drainage measures are in place, site preparation should begin with clearing and grubbing all vegetation and topsoil from the proposed roadway alignment. Stumps and taproots should be cut off at least 2 ft below planned pavement subgrades. Then, all topsoil and organic-laden sand should be stripped. Based on the hand auger borings, we estimate that a topsoil stripping depth of 6 in to 12 in. will be required; however, the topsoil thickness varies across the site and more or less stripping may be required in isolated areas.

Following stripping and prior to fill placement, the geotechnical engineer should evaluate the exposed subgrade along the proposed roadway alignment. This evaluation should include proofrolling with a fully loaded tandem-axle dump truck (or similar equipment judged suitable by the geotechnical engineer in the field) to detect any unstable areas. Any areas that pump or rut excessively should be undercut and replaced with controlled fill. Undercutting should be observed by the geotechnical engineer to confirm that all unsuitable materials are removed and that suitable materials are not over-excavated.

Subgrade Improvement and Stabilization. Sand was encountered beneath the topsoil along the western end of the proposed roadway alignment, where SB#1, SB#2 and SB#9 were performed. We do not anticipate that subgrade improvement will be necessary where sands are present. However, based on our hand auger borings, clays are located directly beneath the topsoil across most of the roadway alignment. In these areas, we expect that subgrade improvement and stabilization will be inevitable.

The easiest and most conservative subgrade improvement method is undercutting. Two other alternatives for subgrade improvement that will reduce the amount of required undercutting are the use of a geotextile or lime stabilization. Decisions regarding subgrade stabilization methods (i.e., undercutting/replacement and reinforcing with a geotextile) can be made most effectively during construction, since their effectiveness is highly dependent on soil and subgrade conditions at the time of construction. Additionally, a laboratory testing program would be necessary to determine the effectiveness of lime stabilization for this project. Undercutting/replacement, the use of geotextiles and lime stabilization are discussed subsequently.

We expect that a significant amount of shallow ($\leq$2 ft) undercutting will be necessary. Based on the results of our hand auger borings, we recommend you establish an undercutting "budget" based on the information in Table 3.

Table 3. Potential Undercutting Depth for Percent of Roadway Length

Potential Undercutting Depth	Percent of Roadway Length
Little or none	10%
1 ft or less	15 to 20%
1 ft to 2 ft	75%

We caution that conditions will vary between the hand auger boring locations. Actual undercutting quantities should be determined in the field at the time of construction by the geotechnical engineer. We note that the extent of required undercutting will depend heavily upon climatic conditions during construction, the aggressiveness of the earthwork schedule, and most importantly, the contractor's ability to maintain sufficient site drainage and limit the disturbance of temporarily wet subgrades. Undercutting quantities may be lessened if construction occurs during a period of relatively dry weather. During periods of heavy rainfall, undercutting quantities may be limited by maintaining adequate site drainage and prohibiting heavy, rubber-tired equipment from travelling on exposed, wet subgrade soils.

It may be possible to reduce the amount of required undercutting by reinforcing moderately weak pavement subgrade soils with a geotextile or Geogrid™, such as Tensar BX1100 or equivalent. We reiterate that decisions regarding the use of geotextiles to stabilize the subgrade should be made during construction. However, for preliminary estimating purposes, we estimate that geotextiles will be needed over 15 to 20 percent of the roadway subgrade.

Another option for reducing undercutting may be lime stabilization or lime modification. The application of lime could stabilize (dry and strengthen) the near-surface clayey soils, thereby, limiting undercutting. Lime usually reacts with most plastic soils containing clay that have a minimum plasticity index (PI) of 10 and fines content of 25% (percent of clay and silt particles passing the No. 200 sieve). The near-surface clays at this site appear to meet this criteria and should react with lime. Laboratory testing would be required to determine the amount of lime required to stabilize or modify the site soils. We anticipate that in areas where lime stabilization/modification is performed, minimal (if any) undercutting would be required. We have performed laboratory and quality assurance testing for several low country projects where lime stabilization was successfully used.

Controlled Fill. Any new fill placed in the proposed roadway area should meet the criteria outline for controlled fill. Controlled fill material should have a liquid limit less than 40 and a plasticity index less than 5 and contain no more than 35% fines (material passing the No. 200 sieve) by weight. Controlled fill should have a maximum dry density (ASTM D 1557) of at least 100 pcf. The soil should be relatively free of organics, deleterious matter and elongated or flat particles, which may be susceptible to degradation. All controlled fill should be compacted to at least 95% of the modified Proctor maximum dry density.

Construction Observations and Testing. An experienced engineering technician, under the direction of a geotechnical engineer, should perform in-place density testing during all fill placement to confirm that the contractor's method can achieve the specified compaction. Additional lab and field testing should be performed to confirm that the fill meets the specified requirements. We recommend a density test frequency of at least one test per 5,000 SF of each fill lift along the proposed roadway alignment. The geotechnical engineer should perform subgrade evaluations and observe all proofrolling and undercutting operations.

Pavement Recommendations. We have performed pavement design analyses for flexible asphalt pavement. The pavement sections described herein were designed using the proprietary software DARWin™ 2.01[5]. Strength testing was not performed on the subgrade soils in the proposed roadway area. We reviewed the results of the subgrade strength testing performed by GeoMetrics, Inc. during their 2000 preliminary exploration for the western portion of the site. However, their laboratory testing was performed on clayey sand and the results of their California Bearing Ratio (CBR) test indicated a CBR value of 9. This CBR value is typical for predominantly sandy soils. Since clay was encountered by most of the hand auger borings performed along the proposed roadway alignment, we based our pavement analysis on an assumed California Bearing Ratio (CBR) value of 6. This value is typical for clayey soils such as those encountered at this site. However, it should be confirmed with laboratory testing when final grades are established. The recommended pavement sections are as follows:

[5] DARWin™ 2.01 was developed by AASHTO and based on the *AASHTO Guide for Design of Pavement Structures* (1993). The pavement design is based on the AASHTO "structural number" (Sn) system.

PAVEMENT COMPONENT THICKNESS (In.)			Maximum Allowable Traffic (ESAL[6])
Graded Aggregate Base Course (marine limestone)	**Asphaltic Concrete Binder Course (SCDOT Type 1)**	**Asphaltic Concrete Surface Course (SCDOT Type 1)**	
8	2	2	1,195,000
10	2.5	2	1,520,000
12	2.5	2.5	3,770,000

Based on our experience with similar developments, these allowable traffic volumes should be adequate. However, if the anticipated traffic volumes are greater than those listed herein, we should be retained to re-evaluate these recommendations.

All subgrade, base and pavement construction operations and material should meet local design codes, and the minimum requirements of the South Carolina Department of Transportation's (SCDOT) "Standard Specifications for Highway Construction," 2000 edition. The applicable sections are identified as:

<u>Title</u>	<u>SCDOT Specification Section</u>
Subgrade	208 (pp. 173 – 175)
Graded Aggregate (marine limestone) Base Course	305 (pp. 209 – 221)
Hot Mixed Asphalt Pavement	401 (pp. 250 – 298)
Hot Mix Asphalt Surface Course (Type 1)	403 (pp. 302 – 305)
Portland Cement Concrete Pavement	501 (pp. 332 – 381)

The performance of pavements will be dependent upon a number of factors, including subgrade conditions at the time of paving, rainwater runoff and traffic. Rainwater runoff should not be allowed to seep below pavements from adjacent areas. In areas where finished pavement grades are <u>below</u> surrounding finished grades, adequate drainage will be critical to pavement performance. The specific need for permanent drainage (i.e., underdrains or swales) will be

[6] Equivalent 18-kip single axle load (ESAL) over the life of the pavement. As examples, a legally loaded tandem axle tractor-trailer has an ESAL of up to 2.5, while a passenger car has an ESAL of approximately 0.0002.

more evident as grading plans are developed and construction gets underway. However, the need for underdrains should be expected in low areas.

All underdrains should consist of a 4-in. diameter perforated pipe surrounded by a No. 57-sized (or coarser) aggregate. The pipe invert should be at least 18 in. below the pavement subgrade elevation. The underdrain trench should be at least 12 in. wide, and a non-woven geotextile (grab tensile strength greater than 100 pounds) should encapsulate the drainage aggregate. The underdrain should be located under or immediately behind the curb, so that the underdrain aggregate and roadway base aggregate are hydraulically connected by continuous contact. The underdrain aggregate should extend to the bottom of the curb, or up to the bottom of the overlying topsoil if located behind the curb.

Immediately prior to paving, the exposed subgrade should be thoroughly evaluated using proofrolling and any unstable areas should be repaired. Since pavement design typically has low factors of safety, it will be important that the specifications are followed closely during construction to insure long-term performance of the pavements. Our analysis was based on a 15-year design life; however, some isolated areas could require repair in a shorter period of time.

The base course should be compacted to at least 98% of the maximum dry density, as determined by the modified Proctor compaction test (ASTM D 1557). In order to confirm that the base course has been uniformly compacted, in-place field density tests should be performed by a qualified engineering technician and the area should be methodically proofrolled under their observation. The base course and asphalt pavement thicknesses should not be deficient in any area by more than ½ in. and ¼ in., respectively.

CONSTRUCTION MONITORING AND TESTING

We strongly recommend that the owner contract with S&ME to perform construction quality control testing and inspection. We recommend that the excavation bottom for the pump station wet well and gravity sewer lines be evaluated by S&ME to verify bottom stability. Undercutting and stabilization operations along the proposed roadway should be monitored by the geotechnical engineer or engineering technician under his supervision. Testing of the foundations for the proposed water tank should be performed according to our previous recommendations within this report.

LIMITATIONS OF REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

CLOSURE

We appreciate the opportunity to be of service on this project. If you have any questions concerning this report, please call.

Sincerely,

S&ME, Inc.

Kasey T. McWhorter, P.E.
Project Engineer

Forrest W. Foshee, P.E.
Vice President

KTM/FWF/jfc

APPENDIX I

FIGURE A: TEST LOCATION PLAN

CPT SOUNDING LOGS

SPT BORING LOGS

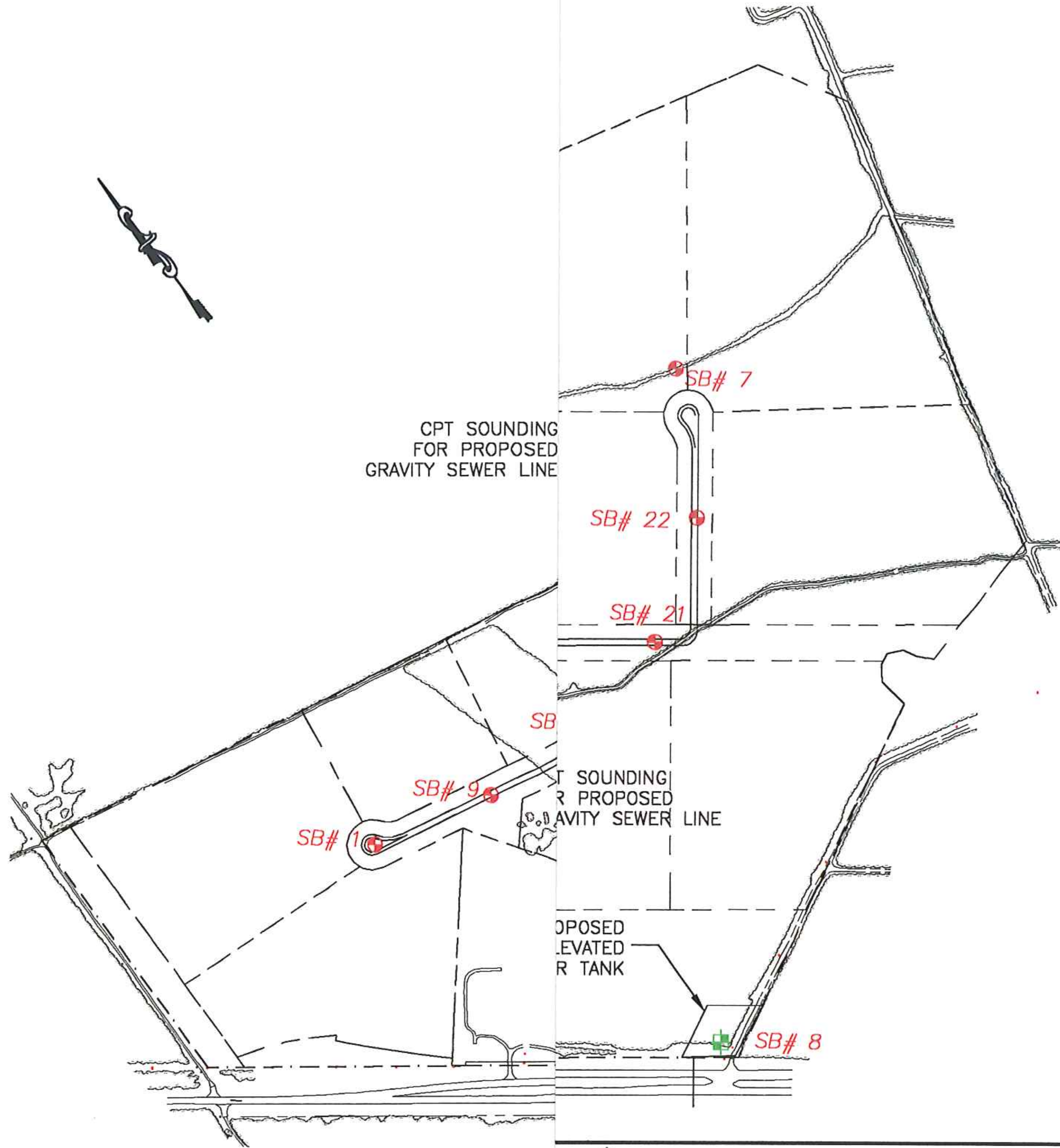
HAND AUGER BORING LOGS

DISSIPATION TEST RESULTS

FIELD TESTING PROCEDURES

FIGURE B: SITE PHOTOGRAPHS

FIGURE C: SCS SOIL SURVEY MAP



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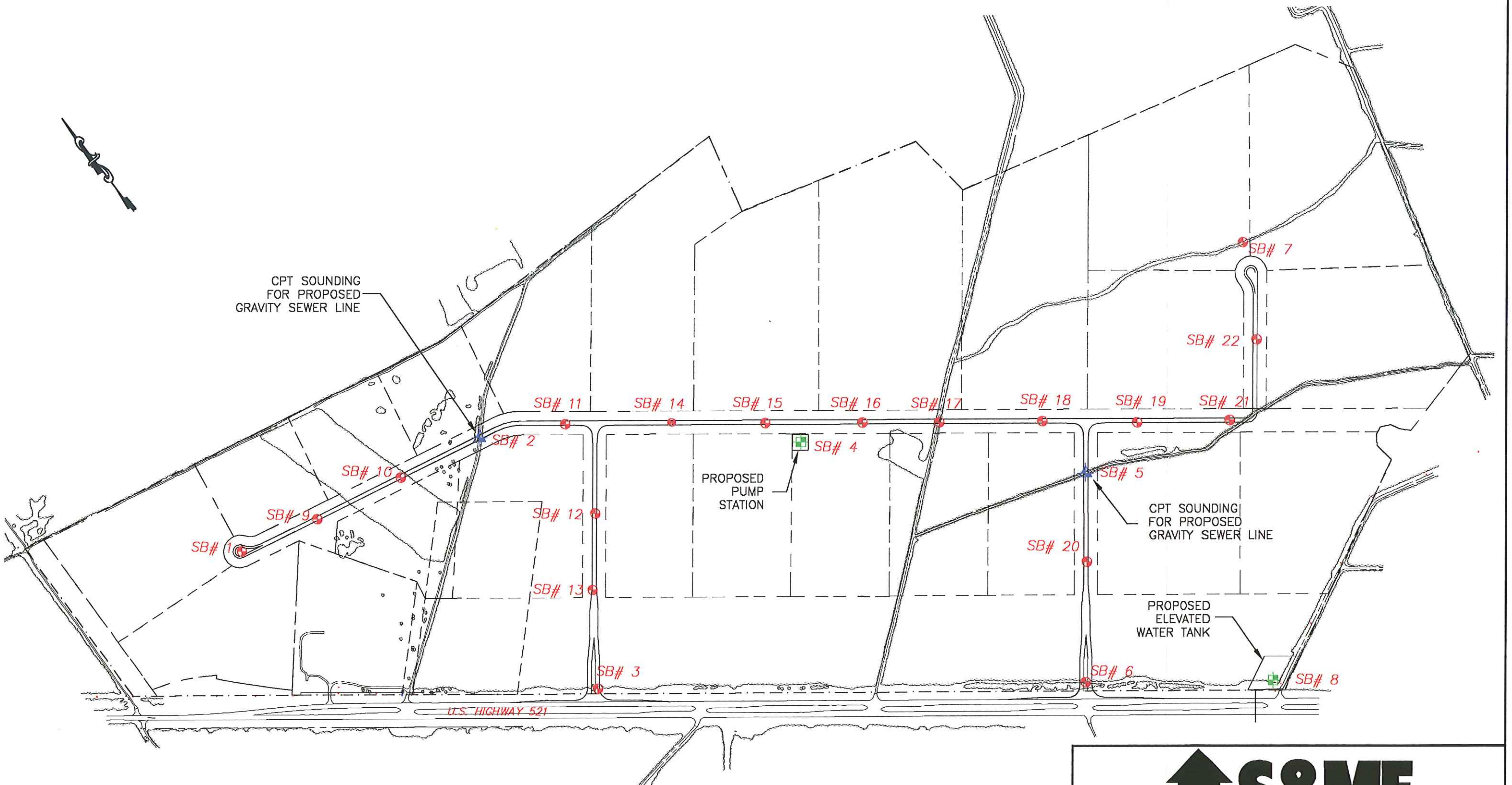
TEST LOCATION PLAN

TOWN COUNTY COMMERCE CENTER

ANDREWS, SOUTH CAROLINA

0'	DRAWN BY: LAJ	APPROVED BY:
2–651	DATE: 10–24–02	FIGURE NO. A

Note: This drawing was provided by Davis & Floyd, Inc and adapted by S&ME, Inc. Test locations are approximate. This drawing should not be used to estimate distances or quantities.



LEGEND

- CPT SOUNDING
- SPT BORING
- HAND AUGER BORING

Note: This drawing was provided by Davis & Floyd, Inc. and adapted by S&ME, Inc. Test locations are approximate. This drawing should not be used to estimate distances or quantities.

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TEST LOCATION PLAN
GEORGETOWN COUNTY COMMERCE CENTER
ANDREWS, SOUTH CAROLINA

SCALE: 1"=600'	DRAWN BY: LAJ	APPROVED BY:
JOB NO. 1131-02-651	DATE: 10-24-02	FIGURE NO. A

PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-1

DATE STARTED:	9/26/02	DATE FINISHED:	9/26/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	GROUND SURFACE AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 3.5 INCHES			
		CLAYEY SAND (SC) gray red brown mottled, fine to medium			
1		- - -			
2		- - -			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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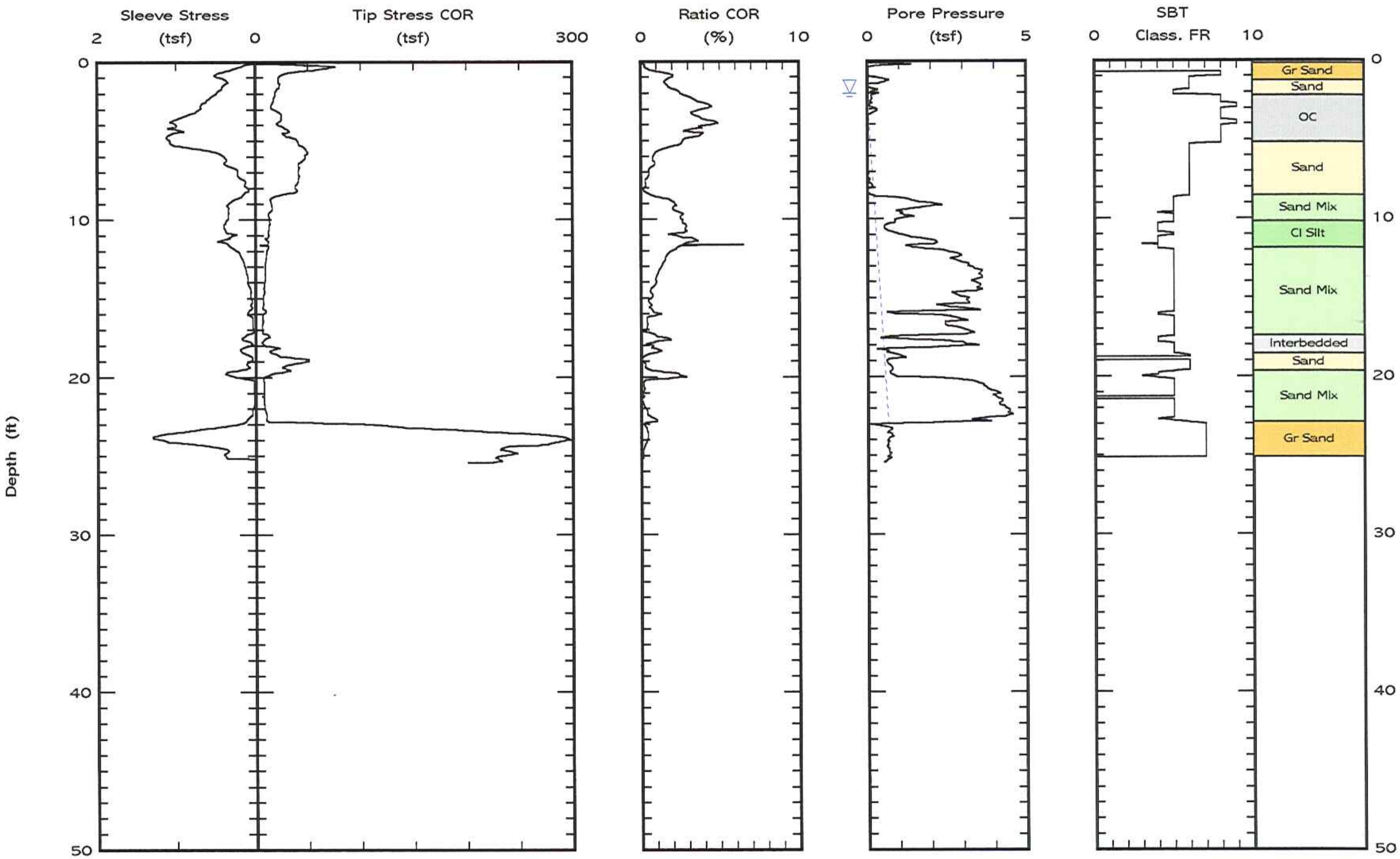
840 LOWCOUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

S&ME Inc. (843)884-0005
840 Low Country Boulevard
Mt. Pleasant, SC 29464

Northing:		Date: 09/Oct/2002
Easting:		Test ID: SB-2
Elevation:		Project: 1131-02-651
Client: Davis & Floyd, Inc.		
Site: Georgetown County Commerce Center		



Maximum depth: 25.47 (ft)

Class FR: Friction Ratio Classification (Ref: Robertson 1990)

∇ Estimated Phreatic Surface

Test ID: SB-2
File: A0900202C.ECP

PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-3
DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.
WATER LEVEL: 0.3 FT AT TIME OF BORING	NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 6 INCHES			
		SILTY CLAY (CL-ML) reddish brown gray mottled; trace sand			
1					
2		- - -			
3		- - -			
4		HAND AUGER TERMINATED AT 4 FEET			

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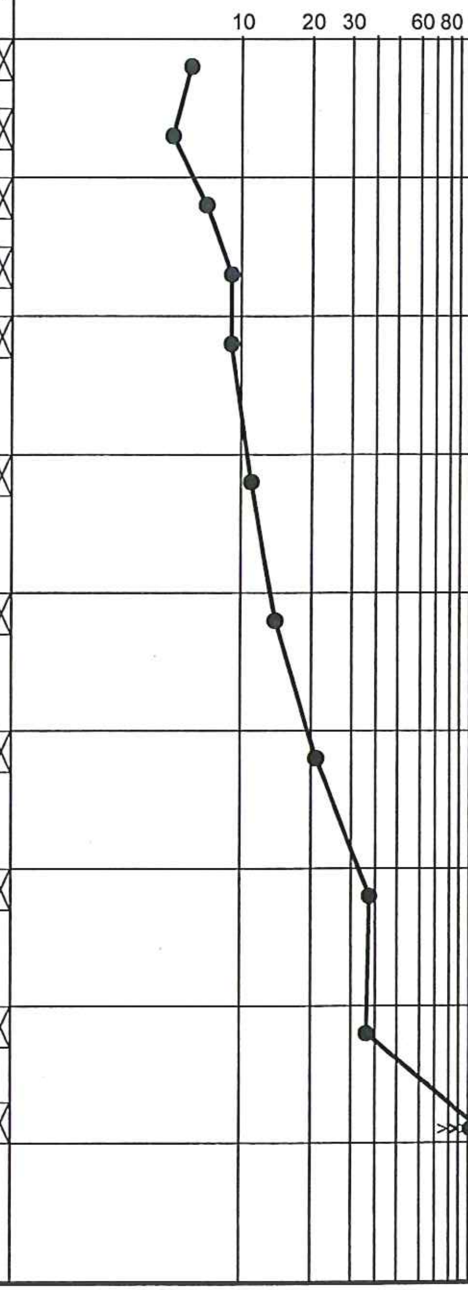
PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	BORING LOG SB-4

DATE DRILLED: 10/9/02	ELEVATION: ft	NOTES:
DRILLING METHOD: MUD ROTARY	BORING DEPTH: 40.0 ft	
LOGGED BY: DWH	WATER LEVEL: 5'3" AT TIME OF BORING	
DRILLER: SCI	DRILL RIG: CME-850 (auto hammer)	

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (ft)	SAMPLE NO/TYPE	N VALUE
0	**TOPSOIL = 8 INCHES**				
	CLAY (CL) firm, reddish brown gray mottled; some organics			S-1	6
	SANDY CLAY (CH/CL) firm, brownish gray			S-2	5
5	---	▽		S-3	7
	CLAY (CL) stiff, gray			S-4	9
10				S-5	9

15	**SAND (SP/SM)** medium dense, gray, fine to medium; slightly silty			S-6	11
20	---			S-7	14
25	**SANTEE LIMESTONE** sampled as medium dense, gray, calcareous sand; fine to medium, high shell content, slightly clayey			S-8	21
30	--- dense, low shell content			S-9	36
35	--- some cemented nodules			S-10	35
	No recovery			S-11	50/ 1"
40	**BORING TERMINATED AT 40 FEET**				

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 02651.GPJ S&ME.GDT 10/21/02

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.

2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

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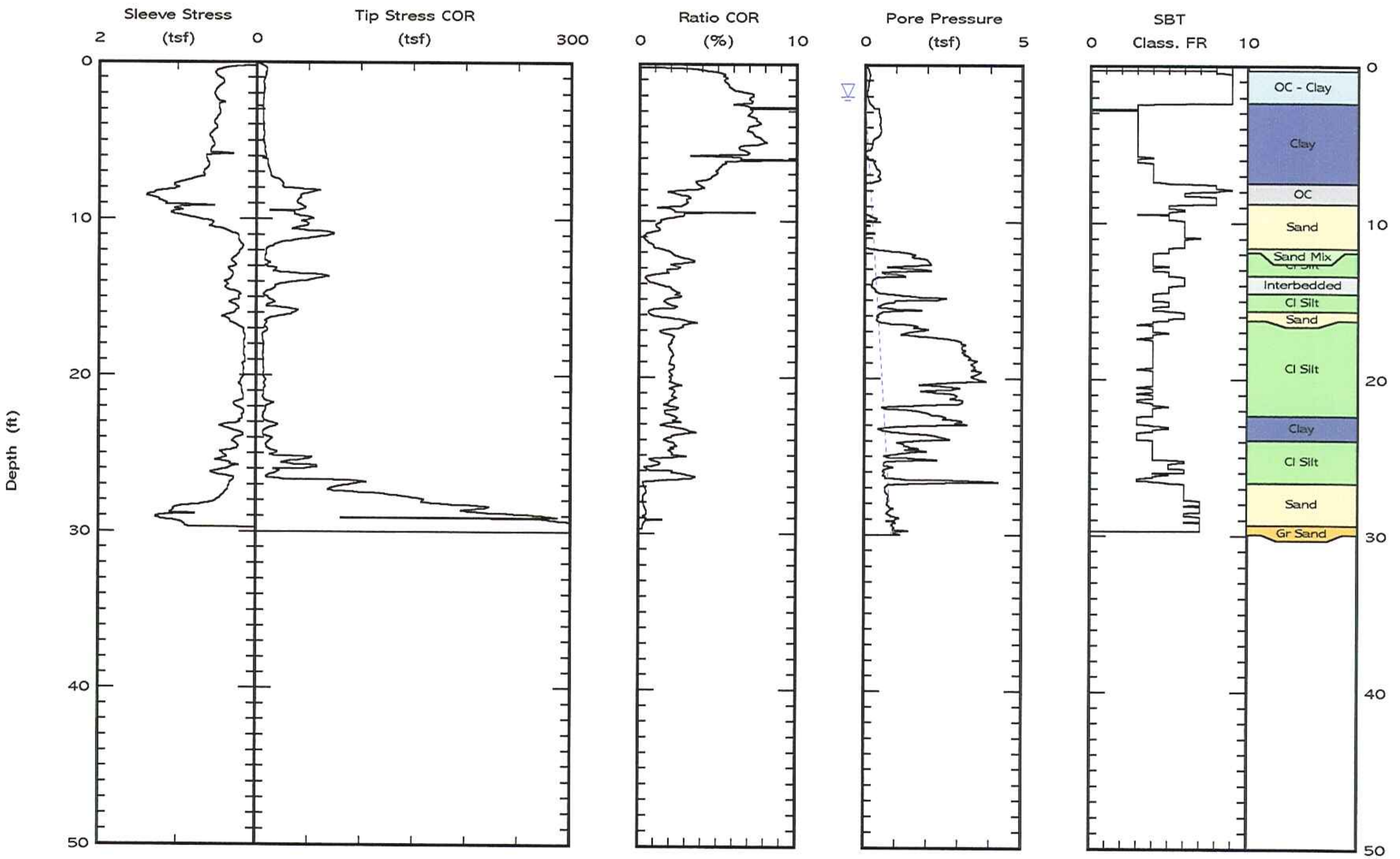
840 Lowcountry Blvd.
Mt. Pleasant SC

S&ME Inc. (843)884-0005 840 Low Country Boulevard Mt. Pleasant, SC 29464	Northing: Easting: Elevation:	Date: 09/Oct/2002 Test ID: SB-5 Project: 1131-02-651
	Client: Davis & Floyd, Inc. Site: Georgetown County Commerce Center	



Maximum depth: 30.00 (ft)

Class FR: Friction Ratio Classification (Ref: Robertson 1990)
∇ Estimated Phreatic Surface

Test ID: SB-5
File: A0900203C.ECP

PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-6
DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.
WATER LEVEL: 2 FT AT TIME OF BORING	NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 2.5 INCHES			
		CLAYEY SILT (ML/CL) gray			
		SILTY CLAY (CL-ML) gray reddish brown mottled			
1					
2		---			
3		---			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651		HAND AUGER BORING LOG: HA@SB-7
DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02	NOTES:
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.	
WATER LEVEL: GROUND SURFACE AT TIME OF BORING		

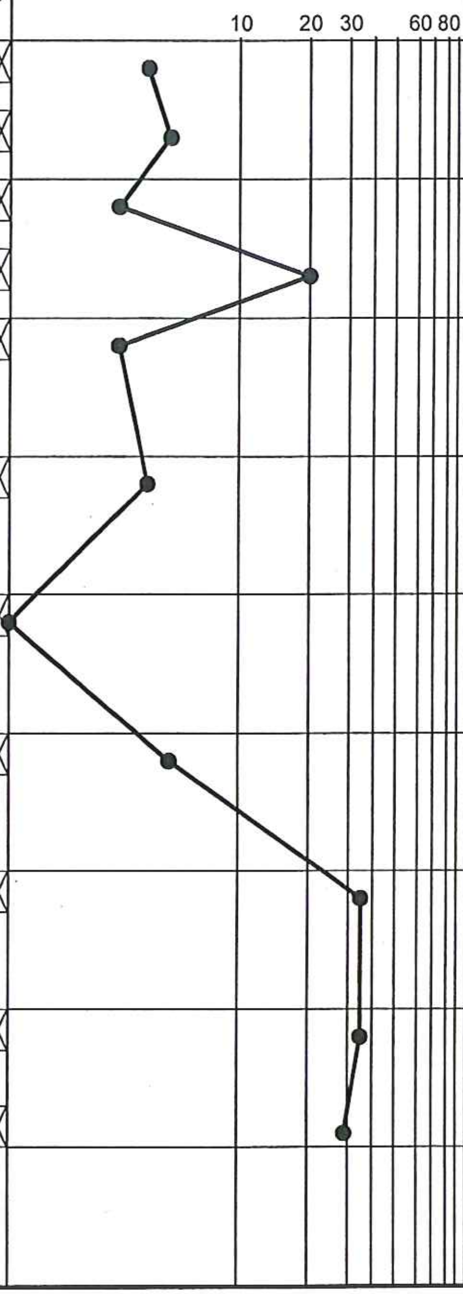
DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 7 INCHES			
		SANDY CLAY (CH/CL) gray, reddish brown mottled			
1					
2		- - -			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	**BORING LOG SB-8**

DATE DRILLED: 10/9/02	ELEVATION: ft	NOTES: f = fines content w = water content LL = Liquid Limit PI = Plasticity Index
DRILLING METHOD: MUD ROTARY	BORING DEPTH: 40.0 ft	
LOGGED BY: DWH	WATER LEVEL: 5'2" AT TIME OF BORING	
DRILLER: SCI	DRILL RIG: CME-850 (auto hammer)	

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (ft)	SAMPLE NO/TYPE	N VALUE
0	**TOPSOIL = 3 INCHES**				
	CLAY (CL) soft, reddish brown gray mottled			S-1	4
	--- firm			S-2	5
5	--- soft	▽		S-3	3
	SILTY SAND (SM) medium dense, brown, fine to medium			S-4	20
10	**CLAY (CL)** soft, dark gray			S-5	3
15	**SANDY CLAY (CL-ML)** soft, dark gray f = 36%, w = 15.3%, LL = 18.5, PI = 8.5			S-6	4
20	--- very soft			S-7	2/ 18"
25	**SANTEE LIMESTONE** sampled as loose, light gray, calcareous, sand; fine to medium, high shell content			S-8	5
30	--- dense; low shell content, with cemented nodules			S-9	34
35	---			S-10	34
40	**SILTY SAND (SM)** medium dense, dark gray, fine f = 23%, w = 31.5%, LL = 22, PI = 0			S-11	29
	BORING TERMINATED AT 40 FEET				

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 02651.GPJ S&ME.GDT 10/21/02

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.

2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-9

DATE STARTED:	9/26/02	DATE FINISHED:	9/26/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	GROUND SURFACE AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 4.5 INCHES			
		CLAYEY SAND (SC) light brown, fine to medium			
1		--- gray red brown mottled			
2		---			
3		---			
4		HAND AUGER TERMINATED AT 4 FEET			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-10
DATE STARTED: 9/26/02	DATE FINISHED: 9/26/02
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.
WATER LEVEL: GROUND SURFACE AT TIME OF BORING	NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 12 INCHES			
1		**SANDY CLAY (CH/CL)** yellow reddish brown mottled			
2		- - -			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-11

DATE STARTED:	10/9/02	DATE FINISHED:	10/9/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	GROUND SURFACE AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 6 INCHES			
		SANDY CLAY (CL) reddish brown and gray mottled			
1					
2					
3					
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-12

DATE STARTED:	9/27/02	DATE FINISHED:	9/27/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	0.2 FT AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 6 INCHES			
		SILTY CLAY (CL-ML) light brown, trace sand			
1					
2		- - -			
		SANDY CLAY (CH/CL) red, yellowish brown mottled			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-13

				NOTES:
DATE STARTED:	9/27/02	DATE FINISHED:	9/27/02	
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.	
WATER LEVEL:	0.3 FT AT TIME OF BORING			

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 7 INCHES			
		SILTY CLAY (CL-ML) grayish brown; trace sand			
1					
2		- - -			
		SANDY CLAY (CH/CL) gray, reddish brown mottled			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-14

DATE STARTED:	10/9/02	DATE FINISHED:	10/9/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	GROUND SURFACE AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 6 INCHES			
		SANDY CLAY (CL) reddish brown and gray mottled			
1					
2					
3					
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-15

DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02	NOTES:
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.	
WATER LEVEL: GROUND SURFACE AT TIME OF BORING		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 7 INCHES			
		SANDY CLAY (CH/CL) red gray yellow mottled			
1					
2					
3					
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	**HAND AUGER BORING LOG: HA@SB-16**

DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02	NOTES:
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.	
WATER LEVEL: 2.5 FT AT TIME OF BORING		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 6 INCHES			
		CLAYEY SILT (ML/CL) red brown; trace sand			
1		- - -			
2		- - -			
3		**SANDY CLAY (CH/CL)** reddish brown gray mottled			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-17
DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.
WATER LEVEL: GROUND SURFACE AT TIME OF BORING	NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 4.5 INCHES			
		SANDY CLAY (CH/CL) gray reddish brown mottled			
1		- - -			
2		- - -			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651		HAND AUGER BORING LOG: HA@SB-18
DATE STARTED: 9/27/02	DATE FINISHED: 9/27/02	NOTES:
SAMPLING METHOD: HAND AUGER	PERFORMED BY: S&ME, INC.	
WATER LEVEL: GROUND SURFACE AT TIME OF BORING		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 11 INCHES			
1		SILTY CLAY (CL-ML) yellowish brown mottled; trace sand			
2		- - -			
3		- - -			
4		HAND AUGER TERMINATED AT 4 FEET			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-19

DATE STARTED:	9/27/02	DATE FINISHED:	9/27/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	0.2 FT AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 8.5 INCHES			
		SILTY CLAY (CL-ML) gray yellowish brown mottled; trace sand			
1					
2		---			
3		---			
4		HAND AUGER TERMINATED AT 4 FEET			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-20

DATE STARTED:	9/27/02	DATE FINISHED:	9/27/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	GROUND SURFACE AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 9.5 INCHES			
1		**SILTY CLAY (CL-ML)** light brown; trace sand			
2		**SANDY CLAY (CH/CL)** gray, reddish brown mottled			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-21

DATE STARTED:	9/27/02	DATE FINISHED:	9/27/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	GROUND SURFACE AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 8.5 INCHES			
		SILTY CLAY (CL-ML) yellow brown; trace sand			
1					
2		- - -			
		SANDY CLAY (CH/CL) red gray, brown mottled			
3		- - -			
4		**HAND AUGER TERMINATED AT 4 FEET**			

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PROJECT: GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA 1131-02-651	HAND AUGER BORING LOG: HA@SB-22

DATE STARTED:	9/27/02	DATE FINISHED:	9/27/02
SAMPLING METHOD:	HAND AUGER	PERFORMED BY:	S&ME, INC.
WATER LEVEL:	0.2 FT AT TIME OF BORING		

NOTES:

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	DYNAMIC CONE PENETRATION RESISTANCE
		TOPSOIL = 11 INCHES			
1		SANDY CLAY (CH/CL) gray, yellowish red mottled			
2		- - -			
3		- - -			
4		HAND AUGER TERMINATED AT 4 FEET			

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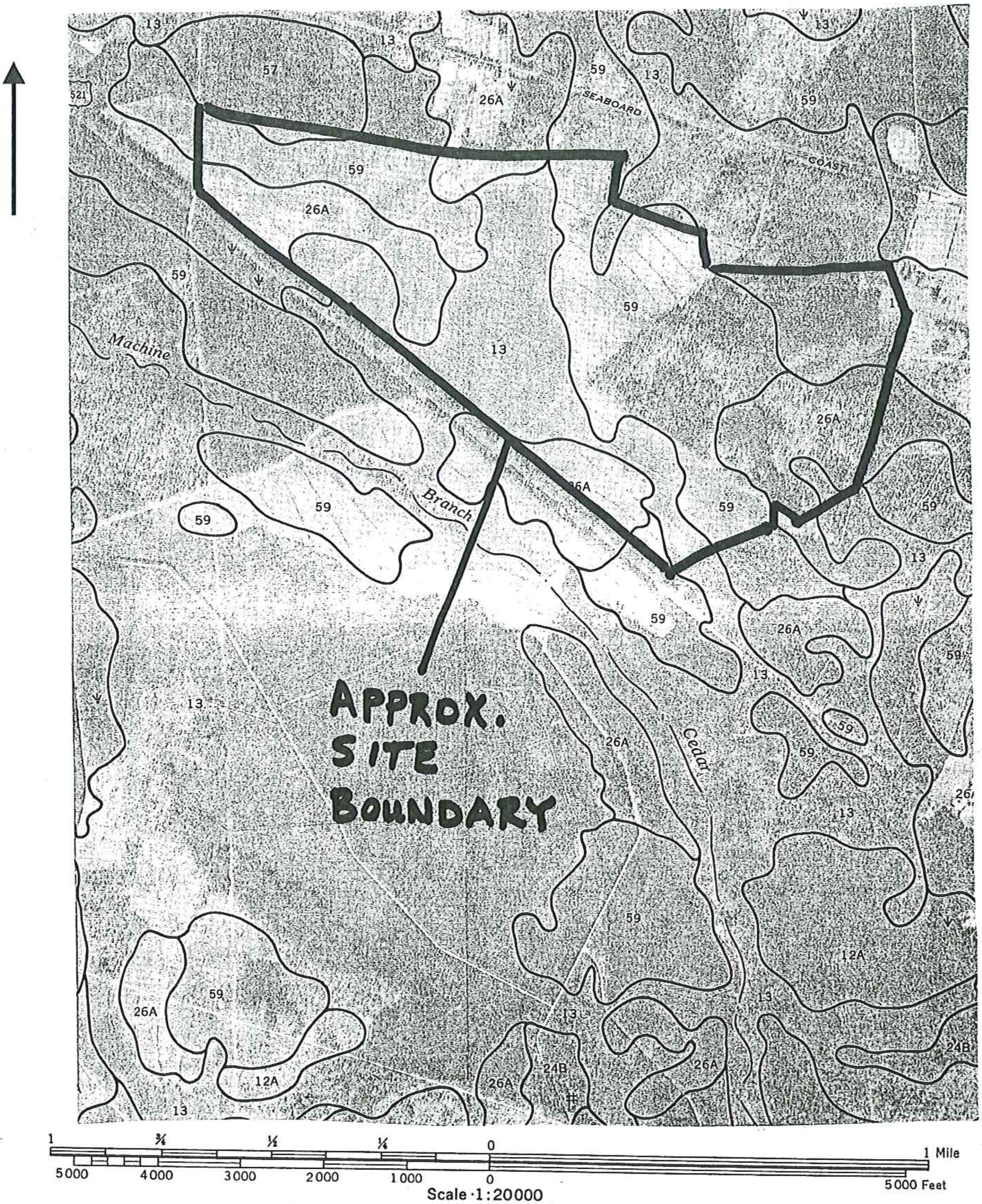


REFERENCE: 1982 SOIL SURVEY OF GEORGETOWN COUNTY, SOUTH CAROLINA

OCTOBER 2002 NOT TO SCALE	S&ME	SCS SOIL SURVEY MAP GEORGETOWN COUNTY COMMERCE CENTER ANDREWS, SOUTH CAROLINA	FIGURE C

Location of SB#5 looking east

Location of SB#5 looking west

Area between SB#5 and SB#18

Area near SB#19

S&ME

Georgetown County Commerce Center
Andrews, South Carolina
S&ME Job No. 1131-02-651

Figure B
Site Photographs

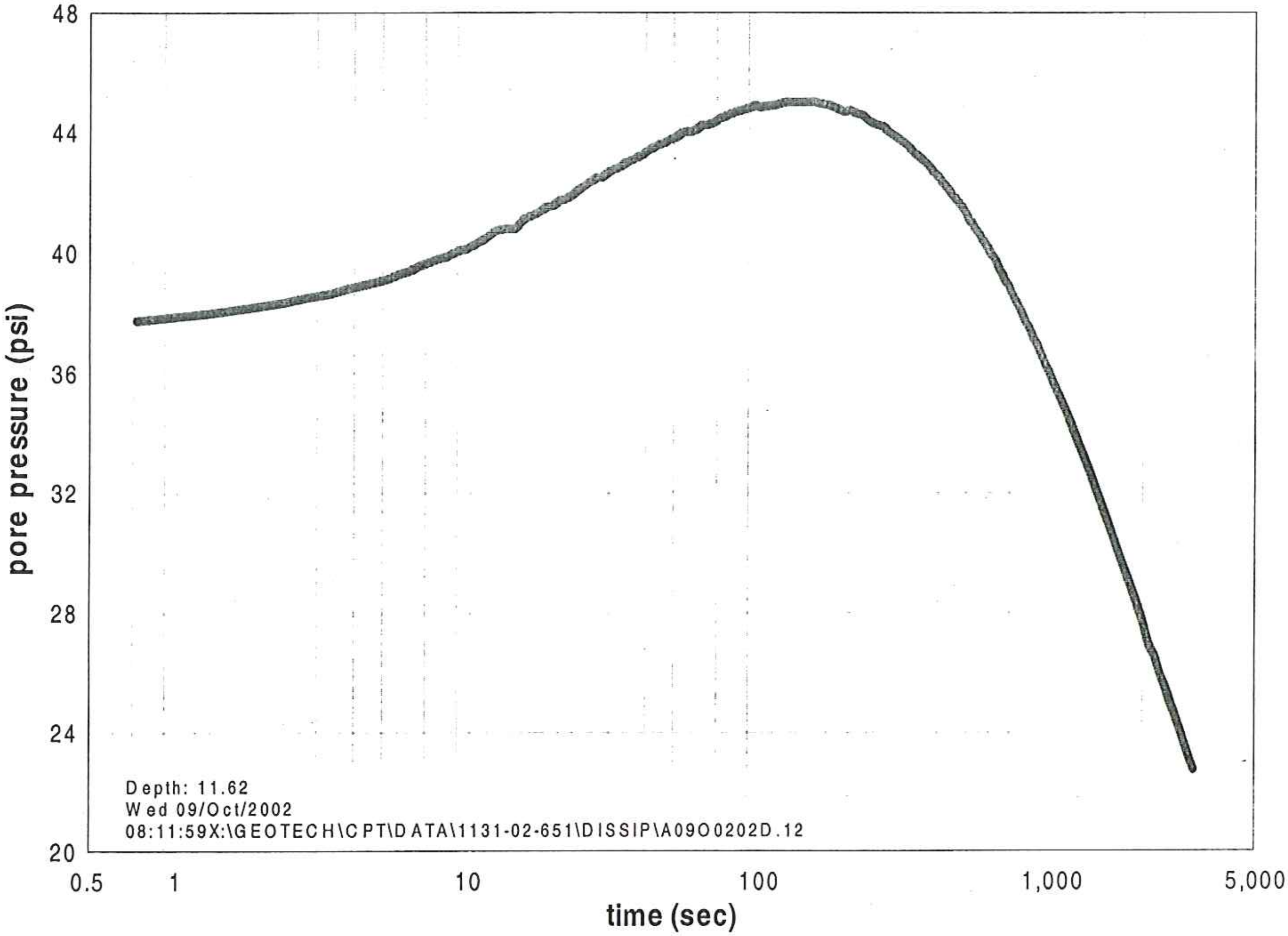
CPTU – PORE PRESSURE DISSIPATION TEST RESULTS

Test ID: SB-2, 10ft
Site: Georgetown Co. Commerce Park
Location: Andrews, SC
Project: 1131-02-651
Client: Davis & Floyd, Inc.

Date: October 9, 2002
Cone Id: 2930.102, 10cm^2

Interpretation Assumptions:
GWT (ft): 2
Depth (ft): 10

Time (sec)	Pore Pressure (psi)	Time (sec)	Pore Pressure (psi)	Time (sec)	Pore Pressure (psi)	Time (sec)	Pore Pressure (psi)



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FIELD TESTING PROCEDURES

Cone Penetrometer Test (CPT) Sounding

The cone penetrometer test soundings (ASTM D-5778) were performed by hydraulically pushing an electronically instrumented cone penetrometer through the soil at a constant rate. As the cone penetrometer tip was advanced through the soil, continuous readings of point stress, sleeve friction and pore water pressure were recorded and stored in the on-site computers. Using theoretical and empirical relationships, the CPT data was used to determine soil stratigraphy and estimate soil properties such as effective stress, friction angle, Young's Modulus and undrained shear strength.

Soil Classifications

Soil classifications provide a general guide to the engineering properties of various soil types and enable the engineer to apply his past experience to current problems. In our exploration, samples obtained during drilling operations are examined and visually classified according to color, texture, and relative density or consistency (based on standard penetration resistance). The consistency and relative density designations are as follows:

SANDS		SILTS AND CLAYS	
N (SPT)	**Relative Density**	**N (SPT)**	**Consistency**
0 - 4	Very Loose	0 - 2	Very Soft
5 - 10	Loose	3 - 4	Soft
11 - 30	Medium Dense	5 - 8	Firm
		9 - 15	Stiff
31 - 50	Dense	16 - 30	Very Stiff
50+	Very Dense	31 - 50	Hard
		50+	Very Hard

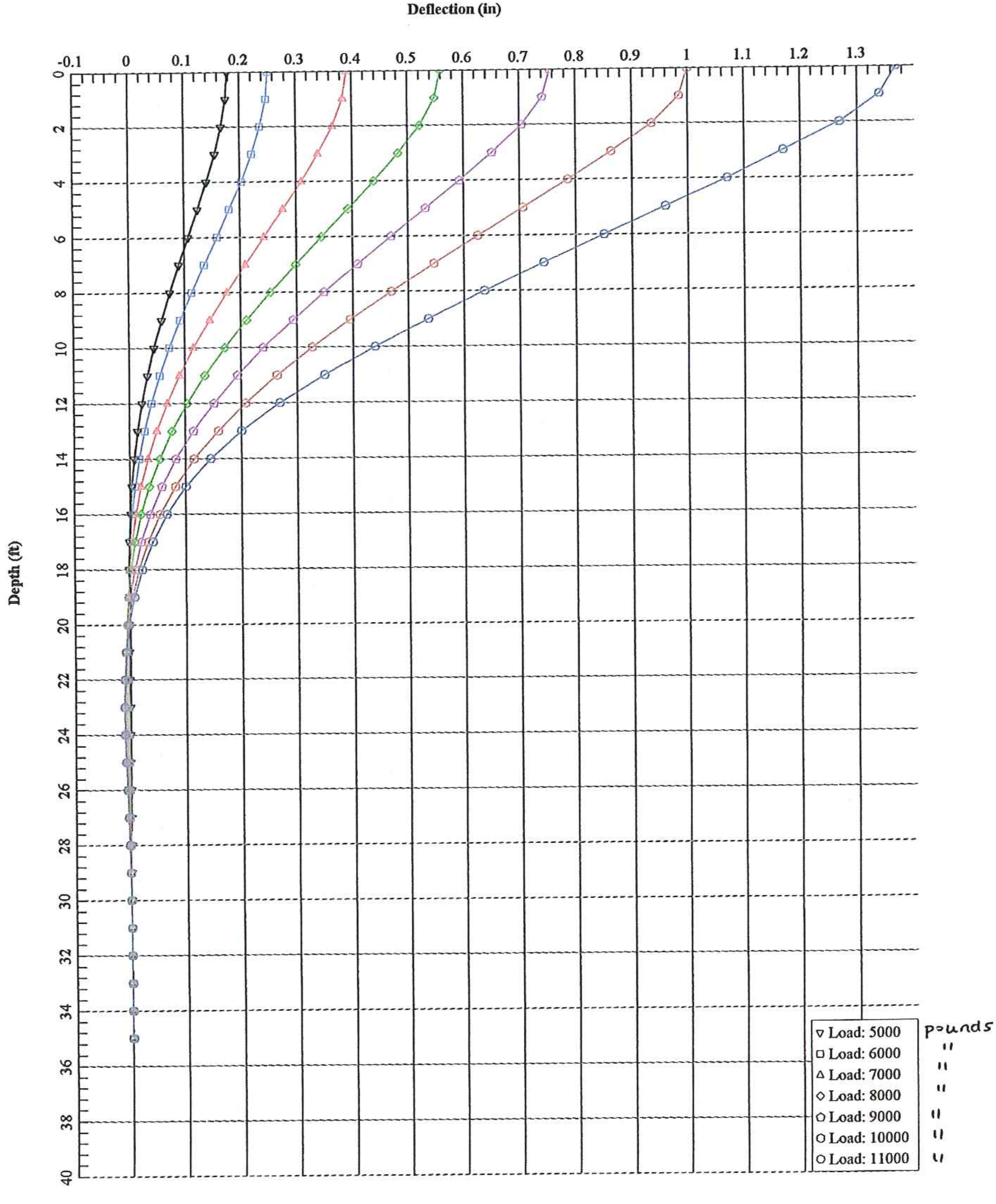
Soil Test Borings

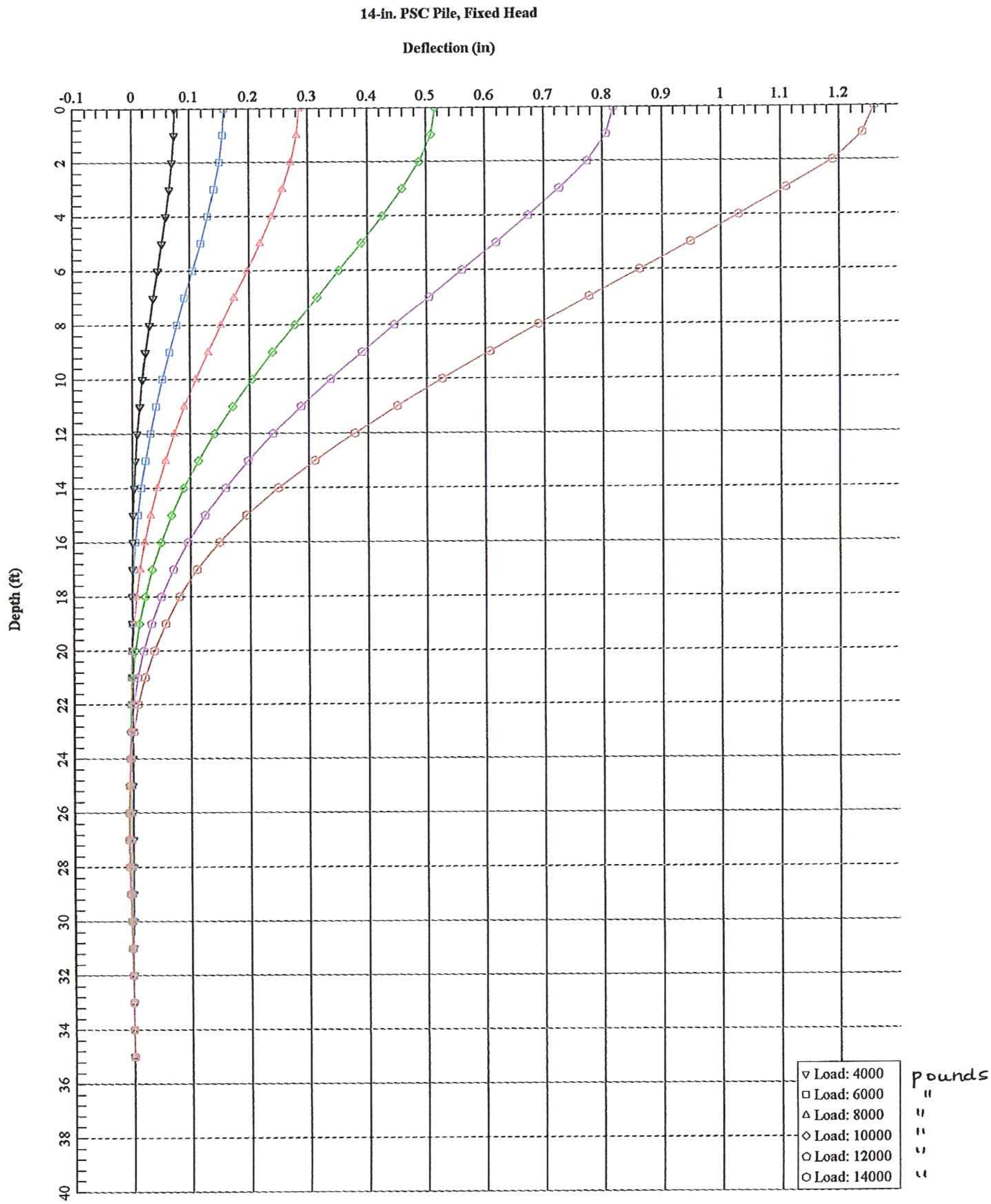
All boring and sampling operations were conducted in accordance with ASTM Designation D-1586. Initially, the borings were advanced by either mechanically augering or wash boring through the soils. Where necessary, a heavy drilling fluid is used below the water table to stabilize the side and bottom of the drill hole. At regular intervals soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-barrel sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140 pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is designated the "Standard Penetration Resistance". The penetration resistance, when properly evaluated, is an index to the soil strength.

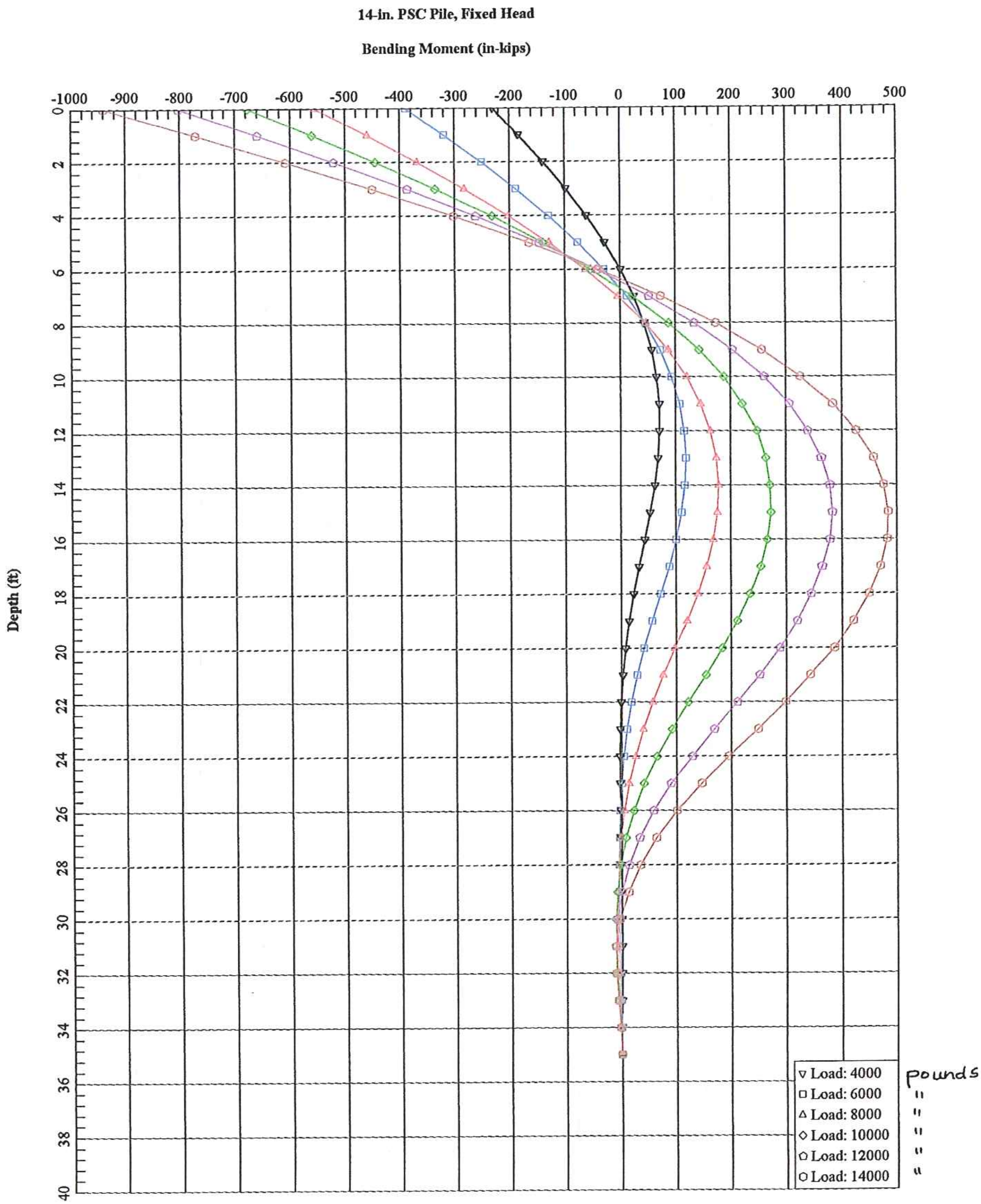
APPENDIX II

RESULTS OF LATERAL ANALYSIS FOR DRIVEN PILES AND DRILLED SHAFTS

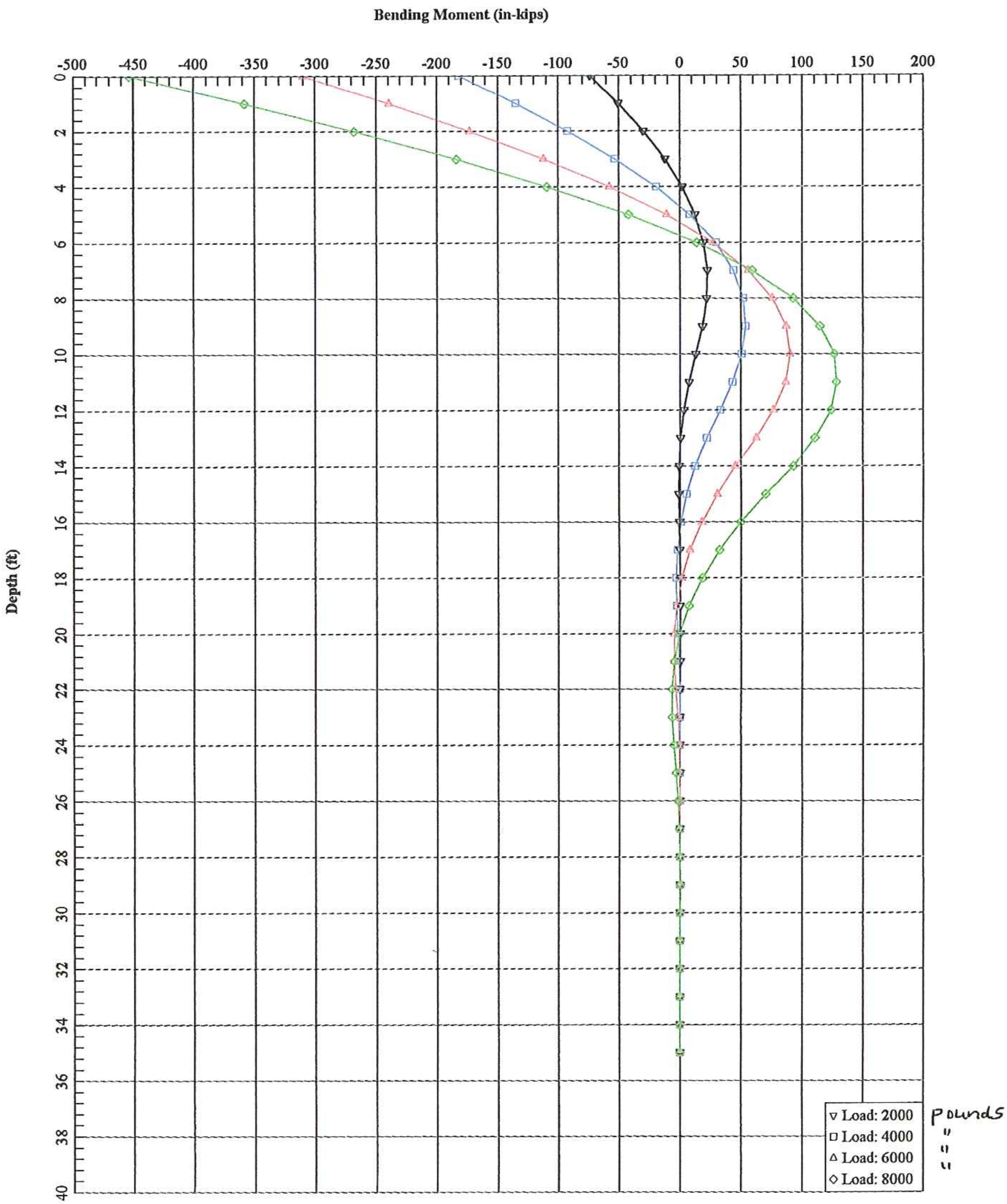
12-in. PSC Pile, Fixed Head

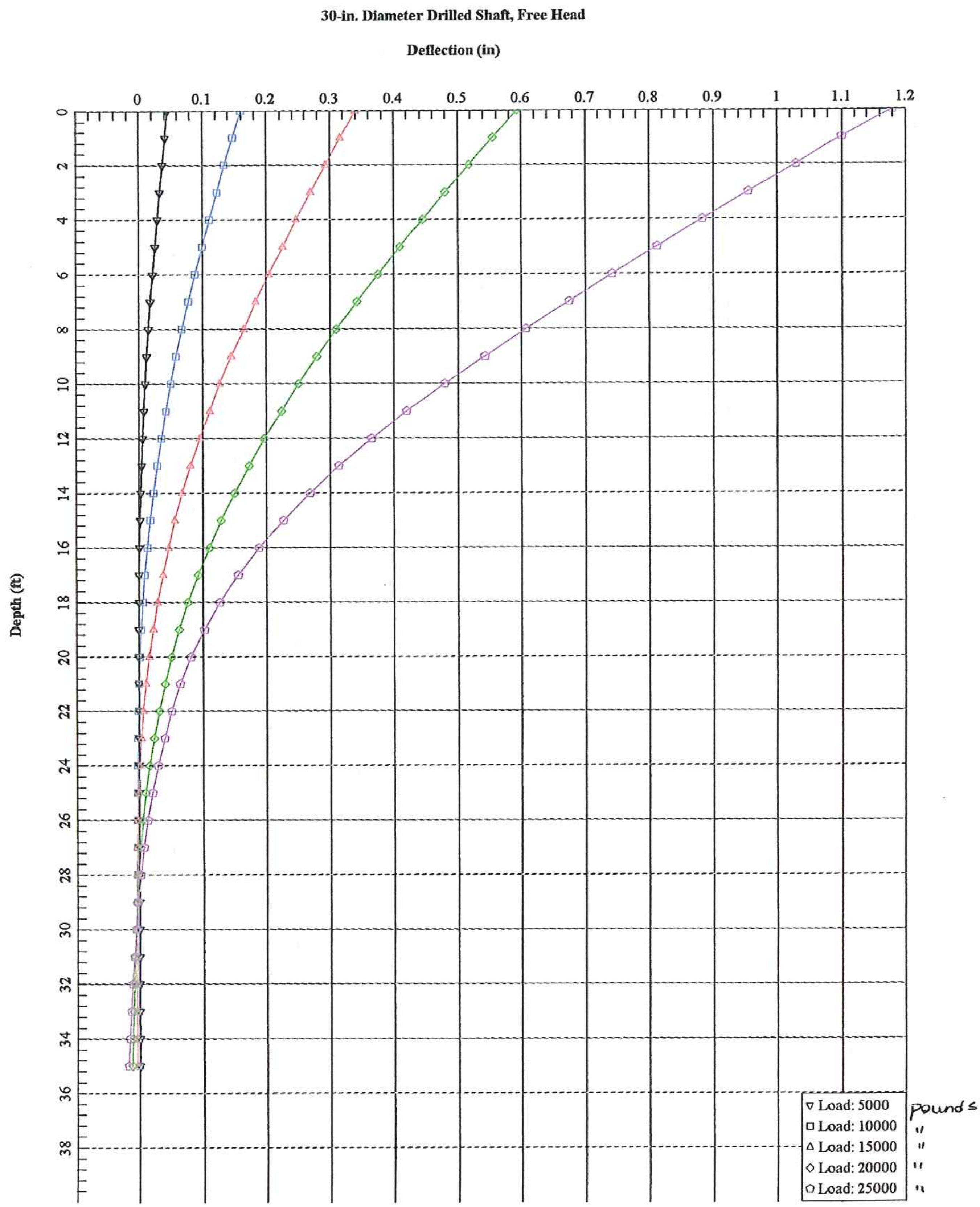



14-in. PSC Pile, Fixed Head
Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Load: 4000
Load: 6000
Load: 8000
Load: 10000
Load: 12000
Load: 14000
pounds
"
"
"
"
"

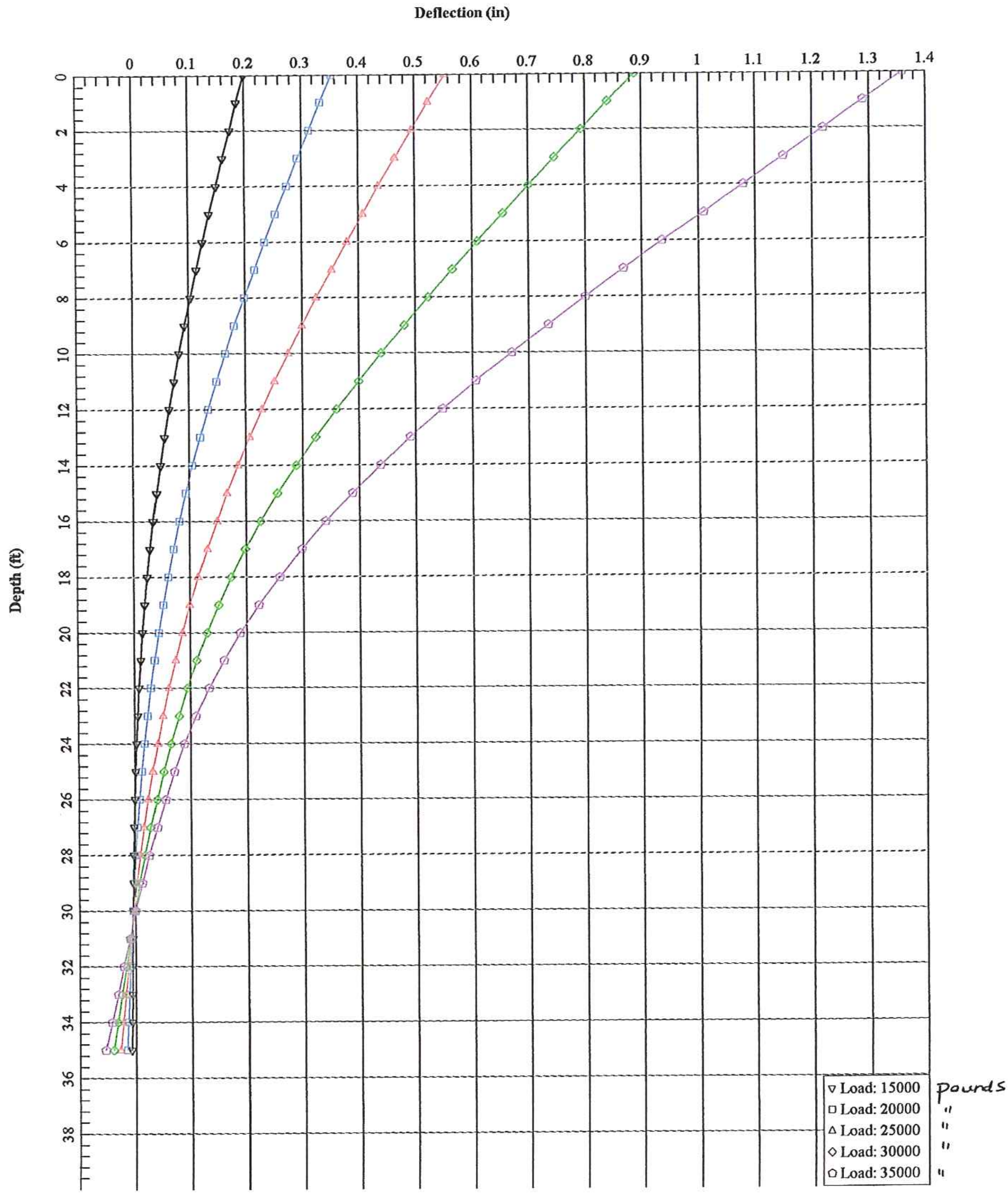

14-in. PSC Pile, Fixed Head
Bending Moment (in-kips)
-1000
-900
-800
-700
-600
-500
-400
-300
-200
-100
0
100
200
300
400
500
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Load: 4000
Load: 6000
Load: 8000
Load: 10000
Load: 12000
Load: 14000
pounds
"
"
"
"
"

10-in. Tip Diameter Timber Pile, Fixed Head

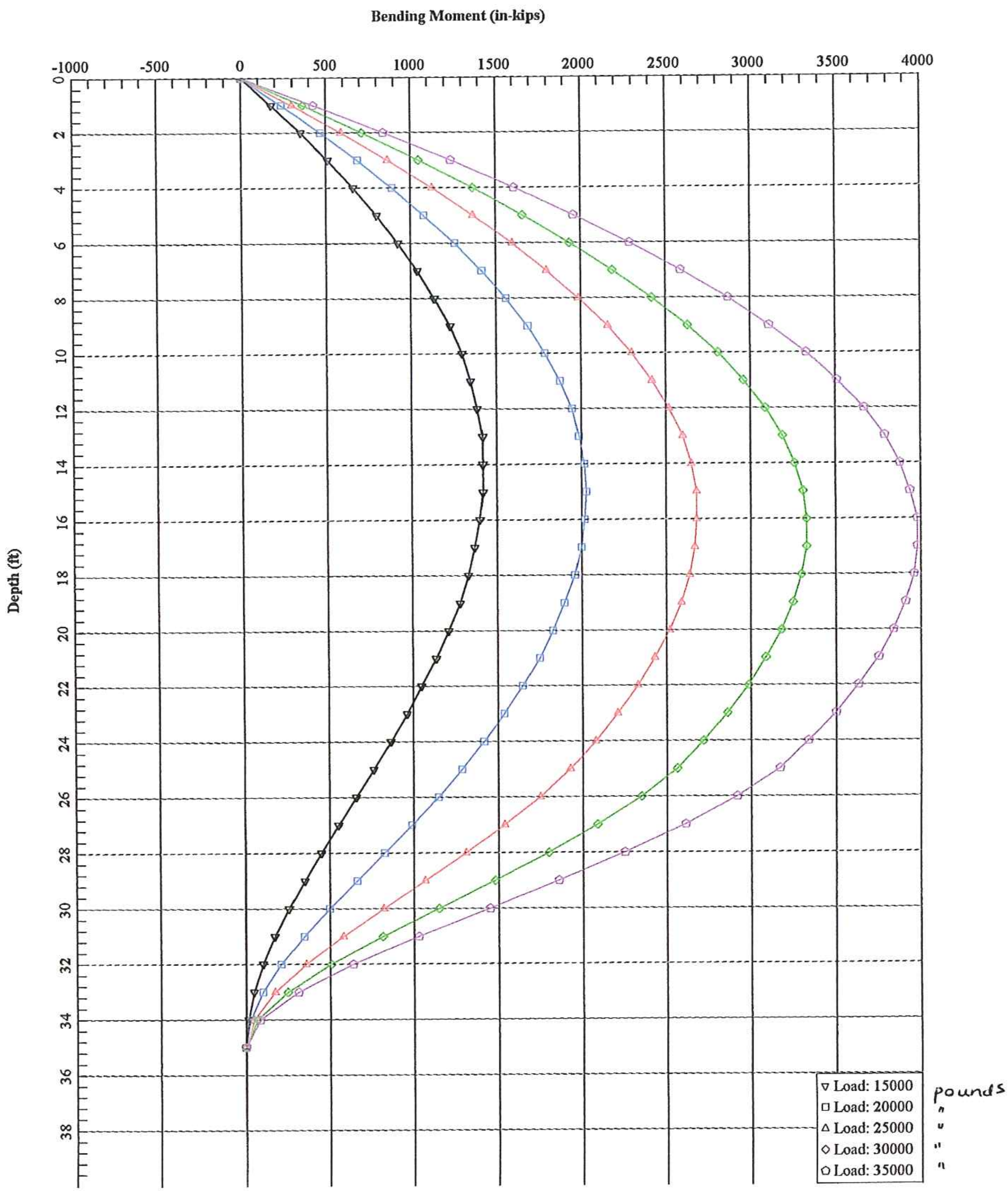


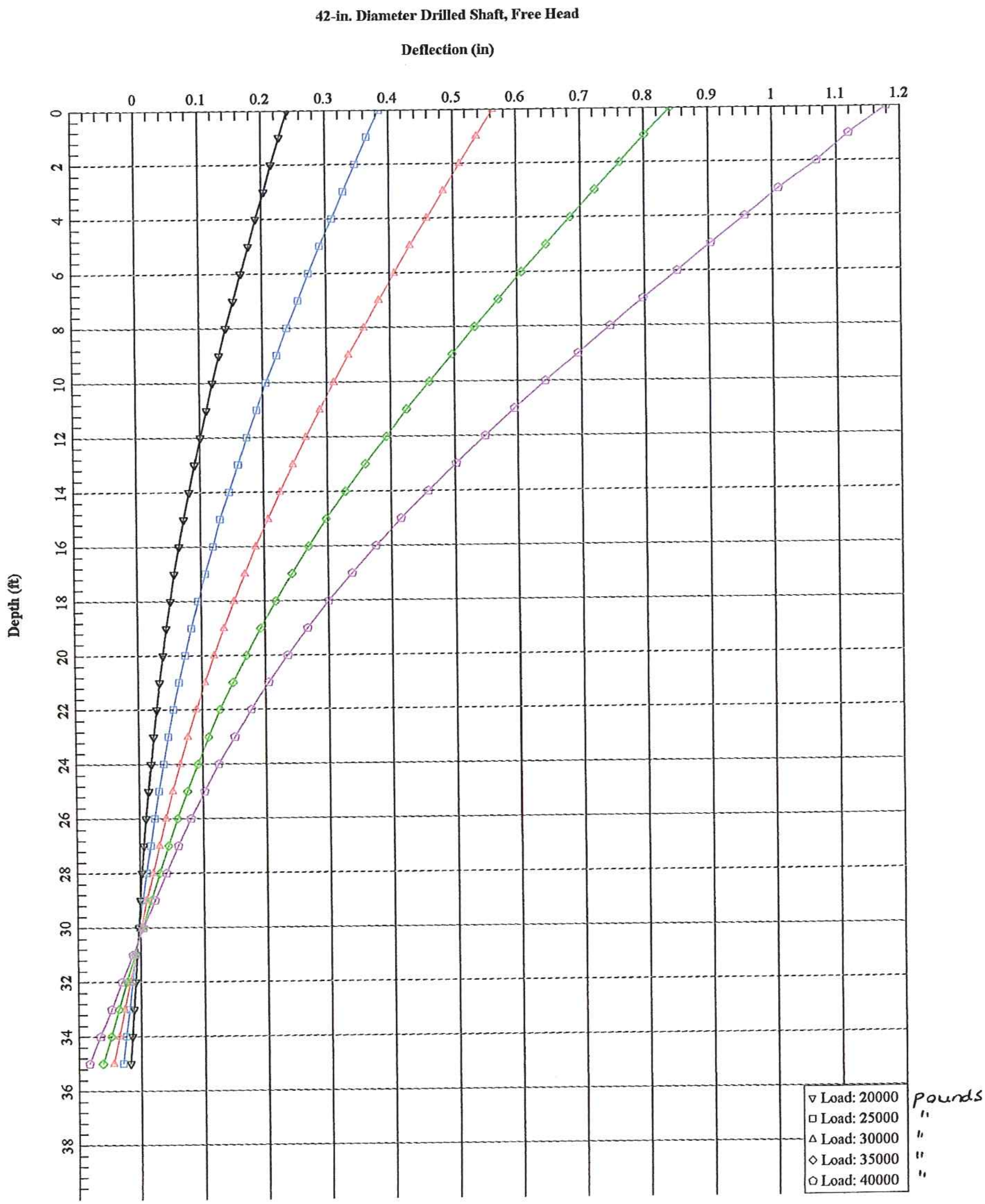

30-in. Diameter Drilled Shaft, Free Head
Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Load: 5000
Load: 10000
Load: 15000
Load: 20000
Load: 25000
pounds

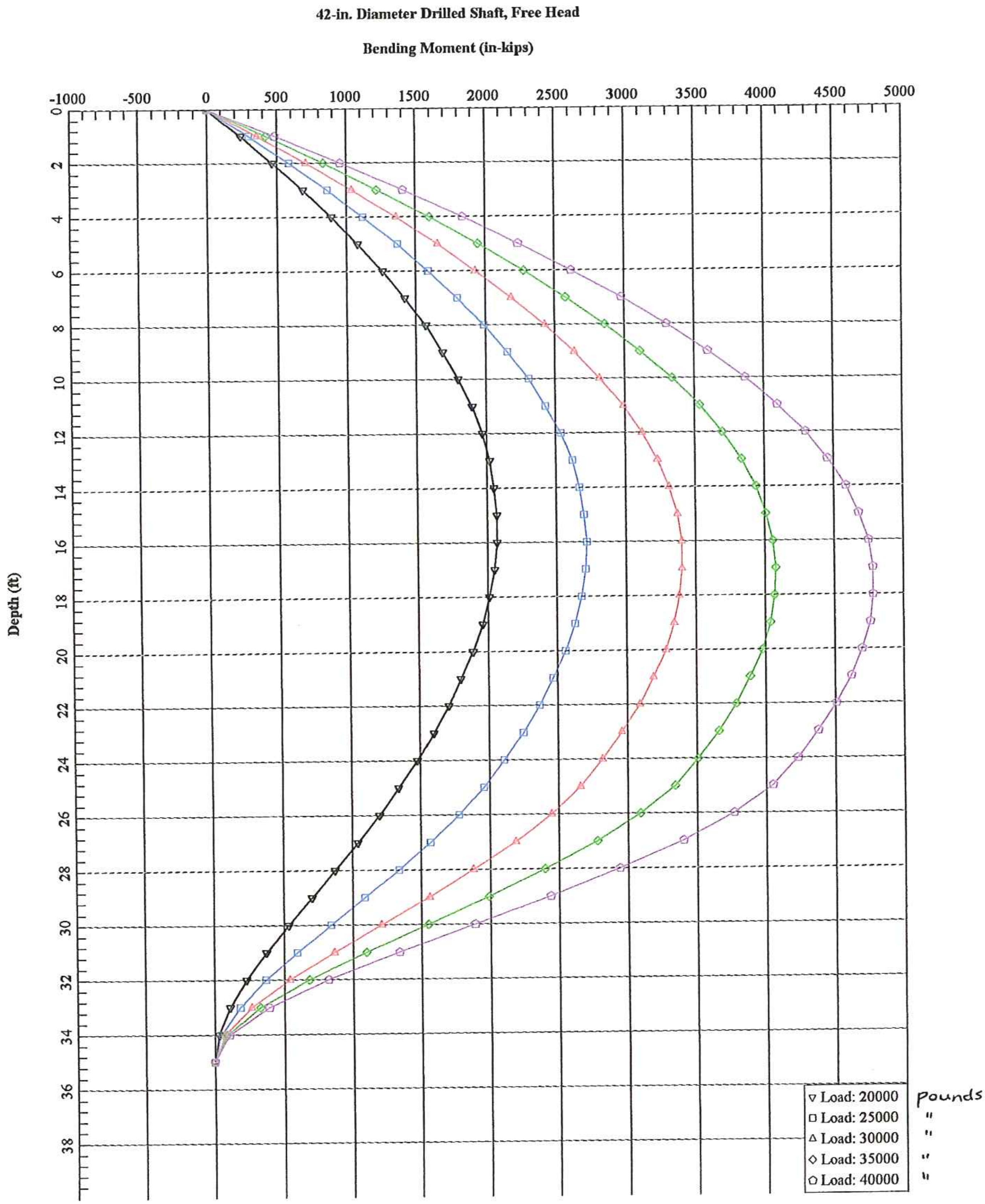
36-in. Diameter Drilled Shaft, Free Head



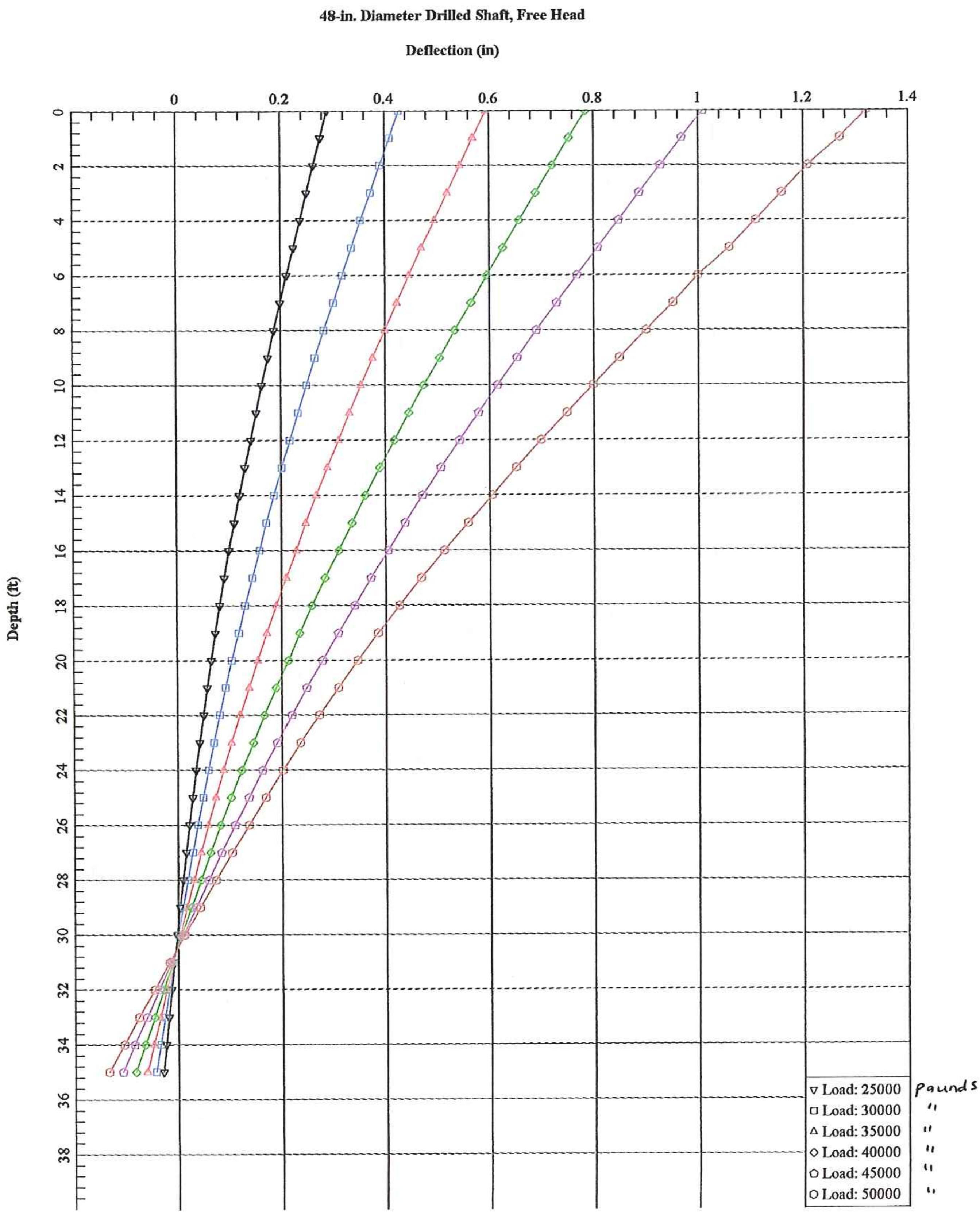
36-in. Diameter Drilled Shaft, Free Head




42-in. Diameter Drilled Shaft, Free Head
Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Load: 20000
Load: 25000
Load: 30000
Load: 35000
Load: 40000
pounds
"
"
"
"



42-in. Diameter Drilled Shaft, Free Head
Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Load: 20000 pounds
Load: 25000 "
Load: 30000 "
Load: 35000 "
Load: 40000 "


48-in. Diameter Drilled Shaft, Free Head
Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Load: 25000 pounds
Load: 30000 "
Load: 35000 "
Load: 40000 "
Load: 45000 "
Load: 50000 "